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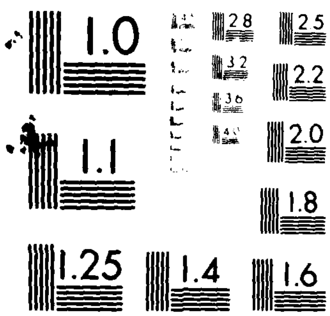

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STRUCTURAL STABILITY EVALUATION PINE RIVER DAM

by

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Final Report

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Prepared for U. S. Army Engineer District, St. Paul
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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

A stability analysis was conducted for a typical interior monolith of Pine River Dam for the following load cases:

(1) Normal operation
(2) Normal operation with truck loading (H15-44)
(3) Normal operation with earthquake
(4) Normal operation with ice
(5) High-water condition.

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20. ABSTRACT (Continued).

A conventional stability analysis (rigid body assumption) was conducted to determine the approximate magnitude of the loads acting on the top of the piles, which support the dam piers.

In order to obtain a detailed stability analysis of the complete monolith to resist the applied loads, the horizontal supporting characteristics of the foundation material were determined by in situ testing using a pressuremeter.

Three NX core holes were drilled through typical monoliths to obtain access to the foundation material. The pressuremeter tests were performed and in situ soils data obtained. The horizontal foundation soil modulus was obtained as a variation with depth into the foundation material and with pressure.

A conservative horizontal modulus of subgrade reaction was obtained and used in a three-dimensional direct stiffness analysis to determine the forces and deflections at the top of the foundation piles. A beam on an elastic foundation analysis was performed and the pressure, moment, and deflection along the length of the most critically loaded pile were determined.

The compressive forces, tensile forces, and deflections predicted for the piles for all load cases were acceptable. However, the shear stresses at the top of the piles and stresses in certain piles were predicted greater than the allowable values. Therefore, it is recommended that a slant-hole, soil anchor system be used to induce compressive forces that will cause the shear and flexural stresses to be below allowables.

From tests on cores, it was determined that the unconfined compressive strength (5600 psi) was adequate. There appears to be some variation in quality of the interior concrete, but it should not be detrimental as long as it does not affect the posttensioning construction. Surface concrete deterioration should be repaired.

After the slant-hole soil anchors are installed and the deteriorated concrete surfaces repaired, the useful life of the dam will be appreciably lengthened.

PREFACE

The evaluation of the stability of Pine River Dam was conducted for the U. S. Army Engineer District, St. Paul, by the Structures Laboratory (SL) of the U. S. Army Engineer Waterways Experiment Station (WES). Authorization for this investigation was given in Intra-Army Order for reimbursable services No. NCS-IA-78-75, dated 23 July 1979.

The contract was monitored by the U. S. Army Engineer District, St. Paul, with principal assistance from Messrs. Jerry Blomker and Roger Ronning. Their cooperation and assistance were greatly appreciated.

The study was performed under the direction of Messrs. Bryant Mather and William Flathau, Chief and Assistant Chief, respectively, SL; and John Scanlon, Chief of the Concrete Technology Division, SL. The structural stability analysis was performed by Dr. Carl Pace and Mr. Roy Campbell. The core logging and writing of the petrographic report was performed under the technical supervision of Mr. Alan Buck by Miss Barbara Pavlov and Mr. Sam Wong. The testing was performed by Mr. Mike Lloyd. The computer programming by Miss Alberta Wade, Automatic Data Processing Center, was appreciated. The core drilling was under the direction of Mr. Mark Vispi, Geotechnical Laboratory, WES. Dr. Pace prepared this report.

Commanders and Directors during the conduct of the program and the preparation and publication of the report were COL John L. Cannon, CE, COL Nelson P. Conover, CE, and COL Tilford C. Creel, CE. Mr. F. R. Brown was Technical Director.

CONTENTS

CONVERSION FACTORS, INCH-POUND TO METRIC (SI) UNITS OF MEASUREMENT

Inch-pound units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
acre-feet	1233.489	cubic metres
feet	0.3048	metres
inch-pounds (force)	0.1129848	newton · metre
inches	0.0254	metres
kips (force)	4448.222	newtons
kips · feet	1355.818	newton · metre
miles (U. S. statute)	1.609347	kilometres
pounds (force) per cubic inch (psi/in.)	0.2714	megapascals per metre
pounds (force) per square inch	6.894757	kilopascals
pounds per inch	175.1268	newtons per metre
square miles	2.589998	square kilometres

STRUCTURAL STABILITY EVALUATION
PINE RIVER DAM

PART I: INTRODUCTION

Background

1. The background material presented below is taken mainly from two publications on Pine River Dam by the U. S. Army Engineer District, St. Paul (1973, 1977) to give pertinent facts in this report concerning Pine River Dam and its construction.

2. The headwaters reservoir system, Figure 1, is one of the oldest projects in the St. Paul District. The initial surveys and investigations were begun in 1867, at a time when the country was being opened up for development and settlement. The projects are old and were designed almost completely on the site. Based on the available data, it appears that the original construction was almost entirely a practical field application in basic engineering. The physical design was performed in the field, and very little documentation was retained. Documentation prior to construction was limited to the amount required to develop the engineering feasibility and requirements for construction, authorization, and funding. Postconstruction documentation was generally limited to reporting quantities, costs, and justification for additional work or study. Construction data since 1915 generally amount to the repair or rehabilitation of existing structures. The data are available, but generally are limited to construction drawings, with little theoretical data. The construction plans give a minimum of data, but they are adequate background for the evaluation of the stability of the structure.

3. Pine River Reservoir (Figures 2 and 3) is one of six Federal reservoirs located in the headwaters region of the Mississippi River, about 120 miles* northwest of Minneapolis, Minnesota, and 90 miles west of Duluth, Minnesota. Four of the reservoirs, including Winnibigoshish,

* A table of factors for converting inch-pound units of measurement to metric (SI) units is presented on page 3.

Leech Lake, Pokegama, and Pine River, were placed in operation in the 1880's; Sandy Lake Reservoir was placed in operation in 1895; and Gull Lake Reservoir became operative in 1912. All six headwater dams were subsequently replaced by concrete or concrete and timber structures in the period from 1900 to 1915.

4. Pine River Reservoir controls the runoff from a 562-square mile drainage area and encompasses about 15 natural lakes, including Cross, Daggett, Little Pine, Rush, Island, Ox, Upper and Lower Whitefish, Big Trout, Arrowhead, Pig, Clamsheel, Bertha, and Upper and Lower Hay Lakes.

5. Originally the headwaters reservoirs were authorized to provide supplemental flow during periods of low flow in the interest of navigation on the Mississippi River at and below the Twin Cities.

6. With the canalization of the Mississippi River below Minneapolis, Minnesota, the demands for storage releases from the reservoir system for navigation have been greatly reduced. Thus, in recent years the reservoirs have been operated primarily for other purposes, including flood control, recreation, fish and wildlife conservation, water supply, water quality improvement, and other related uses. With the exception of Gull Lake, Pine River is the southernmost reservoir. The dam is located at the outlet of Cross Lake on the Pine River about 15 miles above its junction with the Mississippi River, and about 150 river miles above Minneapolis. The dam is at the village of Cross Lake, Minnesota, and is often referred to as the Cross Lake Dam. General reservoir data are presented in Table 1 and pertinent dam data in Table 2.

Control Structure

7. The control structure, Figure 4, is a 150-ft-long, reinforced concrete structure supported on round timber bearing piles. The original structure, which was a rock-filled timber crib structure, deteriorated so quickly that starting in 1905 portions of the superstructure had to be replaced by a concrete and timber structure. The only portion of the original timber crib structure presently remaining is the timber

diaphragm sheeting and round timber bearing piles in the footing and the 23-ft-wide timber apron at the upstream face of the dam. Pine River Dam was originally constructed between 1884 and 1887.

Reconstruction of 1905

8. The reconstruction of 1905 consisted of removal of the timber crib superstructure and replacement with a multiple-bay concrete arch structure on new piers and abutments. New caps were put on the old timber bearing piles and additional new timber piles were driven to support the new and heavier concrete piers and abutments. Mass concrete sections were constructed on new timber pile footings at both abutments. The structure had eleven 6-ft-wide sluiceways that were fitted with hand-operated, low flow control gates, having a maximum opening on each gate of 1.6 ft. Next to the left abutment are two additional 6-ft-wide bays for sluicing logs and to serve as a fishway. Prior to 1972, stop logs located in the 11 bays above the gates and those included in the two left bank sluiceways were used to regulate high flows. The 1905 reconstruction also included removal of the apron floor and replacement with a new timber apron.

Other modifications

9. In 1911, the timber fishway was replaced with a reinforced concrete fishway built on the existing timber piling. In 1948, a contract was awarded for apron and gate repair. Under this contract the timber was removed and all bad pile caps were replaced. At this time voids were found to exist around the pile caps of the entire structure (under piers, abutments, and the apron). Gravel fill was used to fill the voids in the readily accessible areas such as the apron; and hydraulic cement grouting was used to fill inaccessible areas around and under the piers and abutments. The floor areas of the apron between piers were then capped with a 16-in.-thick reinforced concrete slab. A gravel underdrain system with a 6-in. v.c. perforated collector pipe was placed at the underside of the slab. Also, the 32-ft-wide apron downstream of the piers was redecked with a new timber floor in this contract. The slide gates were repaired with new timbers and hardware. In 1971, the timber apron lo' ated on the downstream side of the concrete structure was

replaced with a new 34-ft-wide reinforced concrete apron. The timber piling and timber pile caps remained in place and provided support for the new apron. In 1972, a contract was awarded to remove 12 timber slide gates, hoists, and supporting beams, and to install 11 new 54- by 60-in. sluice gates with operating stands and supporting beams.

Objective

10. The objective of this study is to evaluate the stability of the concrete control structure.

Scope

11. This study was limited to a structural stability evaluation of the concrete control structure with consideration given to foundation and concrete properties. To aid in this evaluation three cores were drilled through the dam into the foundation. One extra core (15.4 ft long) was taken from pier 3 to check the extent of the poor quality concrete. The foundation material was tested in situ in order to determine its supporting capabilities. The concrete cores and foundation material were examined and tested, and the structural stability of the dam was evaluated. The stability analysis was performed in accordance with current Corps of Engineers criteria.

PART II: CORING PROGRAM

12. Since Pine River Dam falls into the classification of a low-head dam, limited coring was performed to obtain properties of the concrete and to obtain an access to the foundation material in which in situ testing was performed.

13. Four core holes were drilled. Four-inch and NX concrete cores were obtained (Part III).

14. Cores PR-P3, PR-P6, and PR-P9 were NX cores taken all the way through the pier.

15. Poor quality concrete was encountered when coring PR-P3 in pier 3; therefore, a second core hole (PR-P3A) of 4-in. size was drilled to check the extent of the poor quality concrete. Core PR-P3A was drilled farther upstream and to a depth of 15.4 ft into the concrete. It was not drilled all the way through the pier. Core PR-P3A was of a better quality concrete; therefore, it is concluded that the concrete in the pier varies in quality.

16. The piers are numbered from right to left looking from upstream to downstream. The location of the core holes in piers 3, 6, and 9 are presented in Figure 5.

17. The core holes were drilled by using a truck-mounted drill rig to core through the roadway and pier. A typical drilling setup is presented in Figure 6.

18. Diamond core bits and 5-ft-long, double-tube, swivel-head core barrels were used to obtain core from the concrete. Holes were drilled into the foundation material; a 60-mm pressuremeter probe was inserted to the desired depths, and pressuremeter tests performed. Alternatively, a split-spoon test was performed and then the hole drilled for a pressuremter test. Two main problems were encountered when trying to perform the pressuremeter test. Gravel continued to fall into and block the core hole. Additional piling had been driven when the concrete piers were constructed, and they were not shown on available drawings, causing some of the tests to be performed too close to a piling to give accurate results. These tests had to be voided.

19. The coring program was oriented toward determining:

a. Depth of deteriorated concrete.

b. Uniformity of concrete with depth.

c. Unconfined compressive strength of the concrete.

d. The foundation material properties by in situ testing, using the core holes as the access to the foundation.

20. The in situ strength of the foundation material is an important factor in the analysis of the stability of the dam, which is supported on timber piles. The drill rig was used to perform pressuremeter tests, standard penetration tests, and to obtain disturbed samples of the foundation material.

21. The coring program was considered a minimum for obtaining representative information on the concrete and foundation material but is adequate for this particular dam. The core holes were not grouted, but capped pipes were used to seal the top openings in order that the core holes could be used in the future for obtaining piezometric data.

22. Pictures of representative concrete cores and of cut sections are presented in Figure 7.

PART III: PETROGRAPHIC REPORT AND CORE LOGS

Samples

23. One 4-in.-diam and three NX size concrete cores were received at the U. S. Army Engineer Waterways Experiment Station (WES) on 29 October for tests and examination. The concrete in Pine River Dam was placed in 1906.

24. All cores were from vertical holes. The cores are described and identified below.

Core No.	Location	El*	Depth ft	Material	Size
PR-P3	Pier 3	1236.32	21.8	Concrete, mortar	NX
PR-P3A	Pier 3	1235.82	15.4	Concrete	4 in.
PR-P6	Pier 6	1236.2	22.1	Concrete, wood, mortar	NX
PR-P9	Pier 9	1236.2	21.0	Concrete, wood, mortar	NX

Test Procedures

25. The four cores were logged at WES. Samples for petrographic examination and for unconfined compressive strength tests were chosen from typical concrete at or near the top, middle, and bottom portions of each core. In some instances the concrete in a certain length of core was highly fragmented, which prevented sampling for physical tests in that area.

26. A piece of core PR-P3 from about the 19.5-ft depth was selected to represent all of the concrete. It was sawed along its axis, and one surface was ground smooth. This surface was then examined with a stereomicroscope. In addition, pieces of different cores were broken, and the fresh fracture surfaces were also examined using a stereomicroscope.

* Elevations (el) cited herein are in feet referred to mean sea level (msl), 1929 adjustment.

27. A cement paste concentrate was prepared from a piece of concrete core PR-P6 at about the 3.5-ft depth. This was done by crushing some of the core and passing it over a 150-μm (No. 100) sieve. The material passing this sieve is considered to be a concentration of cement paste. This paste concentration was then ground to pass a 45-μm (No. 325) sieve and was examined by X-ray diffraction. This was done with an X-ray diffractometer using nickel-filtered copper radiation.

28. Samples of the usually white reaction product found in voids and coating aggregate surfaces in the concrete were collected. The reaction product was examined using a stereomicroscope and as an oil immersion mount using a polarizing microscope.

29. An unusual appearing occurrence of pinkish alkali-silica reaction gel was found in an air void that was adjacent to a granitic coarse aggregate particle when a piece of core PR-P6 was broken. This was from a depth of about 19.7 ft. This material was examined by a scanning electron microscope (SEM) and micrographs were made. The sample was not coated to make it electrically conductive, so a low power setting of 6-kv accelerating potential was used. Therefore, some charging of the sample did occur, but it was kept to a minimum by this procedure.

Results

30. The concrete of core PR-P3 was badly fragmented (Figure 8), while the concrete of the other three cores, PR-P3A, PR-P6, and PR-P9, was generally much more intact (Figures 9-11). Since the larger diameter (4 in.) core (PR-P3A) resampling of pier 3 was more intact than the smaller diameter NX (2-1/8 in.) core (PR-P3) from pier 3, this may mean that much or all of the fragmentation was due to the small NX core size. All of the cores except PR-P3A showed that the original concrete had been overlaid with several inches of nonair-entrained concrete containing smaller aggregate; the contact surfaces of old and newer concrete were loose (Figures 8, 10, and 11). Cores PR-P6 and PR-P9 had been drilled deep enough to recover the wood and mortar base supporting the concrete piers as shown in Figure 8 and 11. Some wood pieces were found as part

of the material recovered below the 21-ft depth in core PR-P6 (Figure 10). Drilling may have destroyed the wood as it did much of the concrete at that depth.

31. Most breaks in the cores were believed to be due to the drilling operation. A stained break at 15.8 ft in core PR-P9 (Figure 11) was the only break believed to predate drilling. Tight cracks at the 3.6-ft depth in core PR-P6 (Figure 10) and the 18.3-ft depth in core PR-P9 (Figure 11) may be old.

32. The original concrete in all of the cores was the same material. It was nonair-entrained concrete containing aggregate of mixed composition and 1-1/2- to 2-in.-max size. The coarse aggregate appeared to be gravel that had some degree of crushing. It was composed of granite and fine-grained, dark, igneous rock particles with lesser amounts of gneiss and quartzite particles. The composition of the fine aggregate was like that of the coarse aggregate except that there were more grains of individual minerals.

33. There was porous portland cement mortar present beneath the wood piling in borings PR-P6 and PR-P9 (Figures 10 and 11). Similar material but with coarser aggregate was present near the bottom of core PR-P3 (Figure 8). This porous mortar and concrete was made of the same or similar materials, except for the absence of coarse aggregate in the mortar, as in the rest of the cores.

34. Material recognized as alkali-silica reaction gel was found in some air voids and on some aggregate surfaces in all four of the cores. Since this gel was most often found associated with the granitic rock particles, this could mean that the granitic particles were, or at least one of, the reactive rocks. The gel appeared as a bluish white, glassy or milky material when examined with a stereomicroscope. It often exhibited shrinkage cracks. As mentioned earlier, some pinkish gel showing an unusual development of layered structure was found in an air void adjacent to a granitic particle of coarse aggregate in core PR-P6 at a depth of about 19.7 ft. The appearance and structure of this gel is shown by the SEM micrographs in Figures 12 and 13. Am immersion mount of some of this gel showed it as fibrous, brownish material with a

refractive index below 1.498 when examined with a polarizing microscope in plane polarized light. Some granitic aggregate particles showed reaction rims. The presence of this gel and the reaction rims on aggregates showed that alkali-silica reaction had occurred.

35. X-ray diffraction of the cement paste revealed compounds that are normally found in hydrated portland cement. These compounds included calcium hydroxide and probably ettringite and tetracalcium aluminate carbonate-11-hydrate (monocarboaluminate) and calcite. Aggregate constituents present as contamination were quartz, potassium and plagioclase feldspar, mica, and possibly amphibole.

Discussion

36. All breaks in the cores appeared to be new except for a break at the 15.5-ft depth in core PR-P9. The closely spaced breaks and fragmentation found in the three NX-size cores were believed to be due largely to the drilling of such small size (mostly NX) cores. The 4-in.-diam (PR-P3A) core showed no signs of fragmentation and only widely spaced breaks even though it was drilled in the same pier as the core that was in the poorest physical condition (PR-P3). While there was clear evidence of alkali-silica reaction in all four cores, it did not appear that this reaction had damaged the concrete, at least not seriously.

37. Although the concrete was not air entrained, it had not been damaged by frost action, which indicated that freezing and critical saturation had not occurred simultaneously.

PART IV: FOUNDATION AND CONCRETE PROPERTIES

In Situ Foundation Testing

38. An estimation of the foundation supporting characteristics for an in-place structure supported by piling is usually based on material properties determined from the sampling of foundation material, transporting and preparing the samples for testing, and testing the samples. The in situ supporting characteristics of the foundation material obtained in this manner are at best approximate. Soil conditions and stress fields can be controlled in the laboratory, but just how faithfully they represent in situ conditions is a matter of conjecture. A further complication at Pine River Dam was that the foundation material was composed mainly of saturated gravel, sand, and silt, which reduced the ability to obtain representative undisturbed samples.

39. For the above reasons, it was considered best to test the foundation material in situ in order to determine the resistance of the soil to horizontal deformation. The pressuremeter method was used to measure foundation deformation properties and obtain a rupture or limit resistance of the foundation material.

Pressuremeter Tests

40. In situ testing to determine the supporting characteristics of a foundation material for a pile substructure has been considered for many years. In 1933 Kögler (Baguelin, Jiziquel, and Shields 1978) wrote about a pressuremeter for obtaining in situ foundation properties. Since before 1965, the pressuremeter has been used in France for the design of building and bridge foundations. Various types of pressuremeters are now being used in the United States; a self-boring pressuremeter shows great promise for future use.

41. In situ testing to determine the resistance of the soil to horizontal displacement is an ideal way to estimate the supporting capacity of material for a pile foundation.

42. The pressuremeter probe was placed within a previously drilled borehole at the desired elevation for testing. Pressure was applied in equal increments and the corresponding volume variations noted at 15, 30, and 60 sec. The data were corrected for calibration, waterhead, etc. The pressuremeter data were used to calculate parameters to be used in analysis to obtain the supporting capability for the pile foundation.

Pressuremeter Field Tests and Results

43. To test the material that supports the pile foundation under Pine River Dam, an access to this material had to be obtained. This was done by coring three NX holes through the dam piers and down to the foundation material. Below each pier, a properly sized hole was drilled in three separate drilling operations. After each drilling operation, the pressuremeter probe was inserted to the desired elevation, and a test was performed. In this manner, three pressuremeter tests were performed at various depths into the foundation material for each test hole.

44. The locations of the probe below the bottom of the pier are presented in Table 3 for each pressuremeter test.

45. Standard penetration (split spoon) tests were also performed in the test holes, and the results are presented in Table 4. The standard penetration values except those for test 1 in hole PR-P3 indicate that the foundation material is compact. Test 1 in hole PR-P3 was at the foundation surface, which was very loose. The second standard penetration test beginning at 5.3 ft below the base of the pier indicated that the material in hole PR-P3 has become compact.

46. Disturbed samples of the foundation material were obtained and transported to WES, Structures Laboratory, for classification. The foundation material under Pine River Dam is made up of silt, sand, gravel, and some clay (Figures 14-23).

47. The main characteristic of the foundation material, which indicates its supporting capability for a pile foundation, is the subgrade modulus and its variation with pressure and depth into the

the foundation. The pressuremeter tests were used to obtain these data. Plots of data from the test holes are presented in Figures 24-37.

48. The recorded pressure had to be corrected to compensate for hydrostatic waterhead in the tubing and for the probe calibration, which gives the resistance to expanding of the rubber membrane. The corrected pressure curves are presented in Figures 24-26 for hole PR-P3 and Figures 31-33 for hole PR-P6.

49. The limit pressure was obtained by plotting pressure versus 1/volume and extrapolating the curves to the pressure at 1/volume = 0. The limit pressure determinations are presented in Figures 25 and 32.

50. The shear modulus (G) (Bagueline, Jiziquel, and Shields 1978) depends not only on the slope of the pressure-volume curve but also on the volume of the probe. The average volume is used in calculating the shear modulus as follows:

$$G_M = \left[535 + \frac{V(I) + V(I+1)}{2}\right]\frac{\Delta p}{\Delta V} \tag{1}$$

$$= \left[535 + \frac{V(I) + V(I+1)}{2}\right]\left[\frac{P(I+1) - P(I)}{V(I+1) - V(I)}\right]$$

51. The deformation modulus, which is something roughly equivalent to Young's modulus, is obtained from the well known relation:

$$G_M = \frac{E_P}{2(1 + \nu)} \tag{2}$$

52. Poisson's ratio is used as 0.33, and the resulting deformation modulus is called the Ménard modulus, E_M .

$$E_M = 2(1 + \nu)G_M \tag{3}$$

$$= 2(1 + 0.33)G_M = 2.66G_M$$

The Ménard modulus is presented in Figures 28 and 35.

53. The subgrade modulus (k) is obtained from the following equations:

$$\frac{1}{k} = \left[\frac{2}{9E_M} B_o \left(\frac{B}{B_o} \times 2.65\right)^{\alpha}\right] + \left(\frac{\alpha}{6E_M} B\right) \qquad (B > 2 \text{ ft}) \qquad (4)$$

or

$$\frac{1}{k} = \frac{B}{E_M}\left(\frac{4(2.56)^{\alpha} + 3\alpha}{18}\right) \qquad (B < 2 \text{ ft}) \qquad (5)$$

where

B_o = reference pile diameter, 2 ft

B = pile diameter

α = rheological coefficient given in Figures 3-48 of Baguelin, Jiziquel, and Shields (1978).

54. The subgrade modulus (k) is presented in Figures 30 and 37.

55. After a representative value of k has been determined, it can be multiplied by the pile diameter to obtain the horizontal modulus of reaction for the pile-soil system. The horizontal modulus of reaction of the soil can be used in the piling analysis to obtain deflections, forces, and moments to use in evaluating the adequacy of the pile foundation.

56. At intermittent times gravel fell into the test hole, which caused difficulty in obtaining properly sized holes and pressuremeter data. Some data were too close to piles and were voided. All data are presented in Figures 24-37.

Piling and Concrete Data

57. The 12-in.-diam Norway Pine pilings, which support the monoliths at Pine River Dam, are approximately 15 ft long. The properties of the Norway Pine are as follows:

Modulus of elasticity (E) = 1.32×10^6 psi

Shear modulus (G) = 0.45×10^6 psi

Allowable compressive stress parallel to grain = 1100 psi

Allowable tensile stress parallel to grain = 775 psi

Allowable average shear stress = 75 psi

Allowable compressive load on a pile = 124 kips

Allowable tensile load on a pile = 0 kips

Average allowable lateral load per pile = 8.5 kips

Allowable moment in a pile = 131,000 in.-lb or 10.9 kip-ft

The properties of Norway wood can be found in many handbooks. One such is presented (Southern Pine Association 1954).

58. The unconfined compressive strengths (average unconfined compressive strength ≈ 5600 psi (Table 5)) are adequate, and since the interior concrete has performed so well for over 70 years, the structure, with some maintenance, can be expected to perform well for many more years.

59. Since the interior concrete is of good quality, the deteriorated surface concrete should be repaired to keep water from entering cracks and accelerating the deterioration of the interior concrete. There are a number of methods of repair that might be used; but, the Upper Mississippi River Headwater Structures are ideal for an economical repair such as:

a. Clean surface concrete.

b. Fill cracks.

c. Paint on a cementitious coating to rehabilitate the surface concrete.

This type repair can be performed rapidly and economically. It is analogous to cleaning, filling cracks, and painting a room in a house. Any local labor could do the work with only common tools.

60. Under some conditions an acrylic-polymer coating of such a composition as listed in Table 6 and Table 7 might be used. Certain acrylic polymers have exhibited good bond and noncracking characteristics when used in ordinary environments. They have also shown good resistance to freezing and thawing environments. The particular polymer to be used should be tested as follows before being used to rehabilitate the surface concrete of the Upper Mississippi River Headwater Structures:

a. Determine the resistance of the coating to cracking during extreme temperature changes.

b. Determine its ability to retain bond capability in freezing and thawing environments.

c. Determine its ability to "breathe," thus allowing water to escape from the interior concrete through the coating, preventing critical saturation of the concrete.

PART V: STABILITY ANALYSIS

Introduction

61. Conventional stability analysis assumes that the base of a structure is rigid in determining the loads on the piles. Conventional stability analysis does not consider the load redistribution due to the pile and structure deformations with consideration being given to the strength characteristics of the soil on the piling system. The monoliths at Pine River Dam are of such size and shape that the assumption of a rigid base is adequate. However, the supporting characteristics of the soil, translation movement of the base, and the deflections of the piles are taken into account by using a modulus of subgrade reaction, which was obtained from in situ test results of the foundation material (Part IV) and used in a direct stiffness analysis.

62. A schematic presenting the geometry of a particular interior monolith of Pine River Dam is presented in Figure 38.

63. Five load cases as follows were analyzed.

- a. Normal operation.
- b. Normal operation with truck loading (H15-44).
- c. Normal operation with earthquake.
- d. Normal operation with ice.
- e. High-water condition.

64. A section view through the pier is presented in Figure 39. A significant fact concerning the pile foundation, as shown by this section, is that three bents (six piles) close to the center of the pier from upstream to downstream are not shown to be in direct support of the bottom of the pier. A plan view of the piling layout for the interior piers is presented in Figure 40. The piling along the center of the interior piers are new piles in relation to the others by the fact that they were added when the timber structure was replaced by a concrete superstructure. The six interior piles will be considered effective in one analysis and ineffective in another. This total analysis will

present a range of values for piling suppport, which will widen one's concept of the stability of the dam monolith of Pine River Dam.

65. There are two other options that will be considered in the analysis of the monoliths.

- a. The applied loads will be taken to el 1215.99 and also to el 1214.32.
- b. The tailwater elevation will be considered at el 1215.12 and also at el 1220.12.

Condition a. considers applied loads to an elevation of the base of the structure (el 1215.99) and also considers the loads to the greater depth of the top of the original piles. Condition b. allows one to consider the effect of a greater tailwater elevation on the uplift pressures, and its effect on monolith stability.

66. If the monoliths are stable under the worst conditions, the elevation is complete; otherwise, some evaluation and decisions will have to be made.

67. The applied loads and moments on the pile system are presented in Figures 41-62. A summary of the forces and moments on the piling system obtained by conventional stability analysis is presented in Tables 8-15. At this point, the adequacy of the pile foundation could not be evaluated because the allowable vertical and horizontal loads based on the supporting capabilities of the foundation material must be known to judge the adequacy of the piles. These allowables were not known for Pine River Dam.

68. To determine the adequacy of the stability of the pile foundation, in situ testing was performed to determine the supporting characteristics of the foundation material. The variation of the subgrade modulus with depth and deformation was obtained. Only three tests were valid (Figures 30 and 37), but the soil under the dam is fairly uniform, and the three tests will allow a conservative subgrade modulus to be selected and used in the analysis of the pile foundation. A conservative constant value, 2000 psi/in., was selected. Due to close pile spacing, the value was reduced to 1500 and 1000 psi/in. for the 8-pile and 14-pile layouts, respectively. The reduction factor was calculated by the following formula.

$$h_a = 0.15 \frac{a}{B} - 0.2 \qquad (3 < \frac{a}{B} < 8)$$

where

h_a = reduction factor

a = center-to-center pile spacing from upstream to downstream

B = pile diameter

If the piling layout is adequate for this analysis, the total dam can be considered adequate in stability.

69. The following guidelines were considered the most efficient and economical to evaluate the adequacy of the piling foundation at Pine River Dam.

a. Core through the monoliths and obtain samples of concrete for evaluation and in the process gain access to the foundation material.

b. Perform in situ pressuremeter tests, as described in Part IV, and use the data to determine the supporting capability of the foundation material.

c. Use the pressuremeter test results to obtain a modulus of subgrade reaction for the foundation material to use in the stability analysis of the pile foundation.

d. Axial, shear, and moments in the pile should be less than allowables based on the properties of the Norway Pine material.

e. The connection of the piling to the structure is assumed to be capable of carrying as much shear load as the pile.

f. The adequacy of the piling, considering the strength characteristics of the foundation material, is based on deflections at the top of the pile. If the deflection of the pile in either a horizontal or vertical direction is less than one-quarter inch, the piling system is considered adequate.

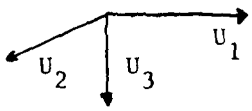
Pile Foundation Analysis Using In Situ Soil-Foundation Properties

70. A general, direct stiffness analysis for a three-dimensional pile foundation was used as has been presented by Saul (1968), which expands the Hrennikoff (1950) method from two dimensions to three. The general solution using this stiffness analysis follows.

71. The forces on a single pile can be equated to the pile displacements by the expression

$$\{F\}_i = \{b\}_i \{X\}_i \qquad (6)$$

The $\{b\}_i$ values are the individual pile stiffness-influence coefficients, called the elastic pile constants. The positive system is as follows:



The $\{b\}_i$ matrix for a three-dimensional system can be defined for the i^{th} pile as:

$$\{b\}_i = \begin{bmatrix} b_{11} & 0 & 0 & 0 & b_{15} & 0 \\ 0 & b_{22} & 0 & b_{24} & 0 & 0 \\ 0 & 0 & b_{33} & 0 & 0 & 0 \\ 0 & b_{42} & 0 & b_{44} & 0 & 0 \\ b_{51} & 0 & 0 & 0 & b_{55} & 0 \\ 0 & 0 & 0 & 0 & 0 & b_{66} \end{bmatrix}$$

The elastic pile constants have meaning as follows:

b_{11} = the force required to displace the pile head a unit distance along the U_1-axis, FORCE/LENGTH.

b_{22} = the force required to displace the pile head a unit distance along the U_2-axis, FORCE/LENGTH.

b_{33} = the force required to displace the pile head a unit distance along the U_3-axis, FORCE/LENGTH.

b_{44} = the moment required to displace the pile head a unit rotation around the U_1-axis, FORCE-LENGTH/RADIAN.

b_{55} = the moment required to displace the pile head a unit rotation around the U_2-axis, FORCE-LENGTH/RADIAN.

b_{66} = the torque required to displace the pile head a unit rotation around the U_3-axis, FORCE/RADIAN.

b_{15} = the force along the U_1-axis caused by a unit rotation of the pile head around the U_2-axis, FORCE/RADIAN.

$-b_{24}$ = the force along the U_2-axis caused by a unit rotation of the pile head around the U_1-axis, FORCE/RADIAN. (NOTE: The sign is negative.)

b_{51} = the moment around the U_2-axis caused by a unit of displacement of the pile head along the U_1-axis, FORCE-LENGTH/LENGTH.

$-b_{42}$ = the moment around the U_1-axis caused by a unit displacement of the pile head along the U_2-axis, FORCE-LENGTH/LENGTH. (NOTE: The sign is negative.)

72. Pile i may be located in the foundation with axes through its origin parallel to the foundation axes. The foundation loads $\{Q\}$ and displacements $\{\Delta\}$ are located with respect to the foundation axes.

73. The forces $\{F\}_i$ due to the pile loads on the pile cap are in equilibrium with a set of forces $\{q\}_i$ at the coordinate center of the pile cap.

74. Equilibrium yields

$$\{q\}_i = \{c\}_i \{F\}_i \tag{7}$$

in which $\{c\}_i$, the statics matrix for a three-dimensional system, is

$$\{c\}_i = \begin{bmatrix} 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & -u_3 & u_2 & 1 & 0 & 0 \\ u_3 & 0 & -u_1 & 0 & 1 & 0 \\ -u_2 & u_1 & 0 & 0 & 0 & 1 \end{bmatrix}$$

where

u_1 = U_1 coordinate of the pile, LENGTH.

u_2 = U_2 coordinate of the pile, LENGTH.

u_3 = U_3 coordinate of the pile, LENGTH.

Foundation stiffness analysis

75. If the piling cap is assumed rigid, then the deflection of the pile cap can be related to the deflection of the piling in the foundation axis coordinates by

$$\{x\}_i = \{c\}_i^T\{\Delta\} \qquad (8)$$

76. The foundation load $\{Q\}$ is distributed to each piling so that

$$\{Q\} = \sum_{i=1}^{n} \{q\}_i \qquad (9)$$

where n = number of piles. The relationships between the foundation load and the pile cap deflections are

$$\{Q\} = \{s\}\{\Delta\} \qquad (10)$$

in which $\{s\}$ is the stiffness influence coefficients matrix for the foundation as a whole. The $\{s\}$ matrix is found by introducing the contribution of each individual pile toward the stiffness of the pile cap. This yields

$$\{q\}_i = \{s'\}_i\{\Delta\} \qquad (11)$$

in which

$$\{s'\}_i = \{c\}_i\{a\}_i\{b\}_i\{a\}_i^T\{c\}_i^T \qquad (12)$$

and finally

$$\{s\} = \sum_{i=1}^{n} \{s'\}_i \qquad (13)$$

where $\{a\}$ is the transformation matrix of force and displacement of the pile (rotated and/or battered) axis to the foundation axis.

77. Once the stiffness matrix is known for the total foundation, the problem is essentially solved and only requires back substitution to find the distribution of loads to the individual piling. It can be noted that the foundation stiffness matrix $\{s\}$ is independent of the external loads.

Loads and displacements

78. The displacements of the pile cap can be found by inverting the foundation stiffness matrix $\{s\}$ and multiplying it by the external load matrix $\{Q\}$ or

$$\{\Delta\} = \{s\}^{-1}\{Q\} \qquad (14)$$

79. Once the foundation deflections are known, the deflections of pile i about its own axes can be found by

$$\{x\}_i = \{a\}_i^T\{c\}_i^T\{\Delta\} \tag{15}$$

80. Finally, the forces allotted to each pile about its axes can be found from Equation 6 where

$$\{F\}_i = \{b\}_i\{x\}_i \tag{6bis}$$

Forces and Deflections of Individual Piles

81. The approach followed in obtaining the forces and deflections on the individual piles was as follows. The modulus of subgrade reaction, the material properties of the pile, and the pile length were used to determine the pile-head stiffness matrix for a single pile, assuming a linear elastic pile-soil system. This pile-head stiffness matrix was obtained by using a finite element computer code (Martin, Jones, and Radhakrishnan 1980) which is a one-dimensional finite element analysis of a beam on an elastic foundation.

82. The pile-head stiffness matrix was then used as input in another computer program that uses the direct stiffness analysis to obtain the forces and deflections of the piles. A beam on an elastic foundation analysis was also performed and the pressures, moments, and deflections along the length of the most critically loaded pile were determined.

83. The analysis assumed that the top of the piles are pinned to the base of the monolith, and that the monolith base is rigid. These assumptions are adequate for the dam construction of Pine River Dam.

84. The results of the three-dimensional, direct stiffness pile foundation analysis are presented in Tables 16-19. The allowable loads on the pile are presented in Part IV.

85. The pressures, moments, and deflections along the length of the pile for the most critical load cases (normal operation with ice and high-water condition) are presented in Figures 63 and 64. The maximum

and minimum stresses in the pile due to the applied loads and moments are 1413 and 1236 psi compression and 724 and 618 psi tension, respectively, for normal operation with ice and the high-water condition, respectively.

86. For only eight piles to effectively support the Pine River Dam monoliths the maximum compressive loads, tensile loads, and deflections of the piles are adequate. The shear load at the top of the piles and the stresses in some individual piles are excessive. To eliminate these overstresses it is suggested that posttensioning anchors be placed in the monoliths.

87. In the posttensioning process, the main cost will be the drilling through the concrete piers and the placement of the tendons; therefore, the maximum lateral load of 20.7 kips minus 8.5 kips allowable per pile will be provided. The 20.7 kips per pile is for the case loading of normal operation with ice with the reference elevation at 1216.2.

88. The main concern with this posttensioning construction is to make sure that it does not overstress any of the piles for normal operation cases. The resultant loading for the normal operation case, normal operation with truck loading, and normal operation with earthquake already is upstream of the centroid of the pile layout. The assurance of no overstress will be accomplished by not posttensioning the tendons to their maximum capacity. Only enough posttensioning will be stressed in the tendons to give some restraint to the monolith; and if the case loading of high-water condition or normal operation with ice do occur, the ability for posttensioning will be present and the steel tendons will take up the stress with very little strain. In this way the posttensioning will only be used to a substantial degree if it is needed; otherwise, the stress will not be induced in the monolith.

89. The posttensioning is figured as follows:

a. Tendons 2 ft apart and 2 ft from each side of pier as shown in Figure 65.

b. Posttensioning per tendon $= \frac{(8)}{2}(20.7 - 8.5)\frac{\sqrt{6.5^2 + 19.13^2}}{6.5}$

$\approx$ 152 kips per tendon.

c. Posttension only 75 kips into each tendon.

d. The component additions are:

(1) Reference el 1217.87

$$F_y = F_H = \left(\frac{6.5}{20.2}\right)(150) = 48.27 \ \frac{\text{kips}}{\text{pier}}$$

$$F_z = F_V = \left(\frac{19.13}{20.2}\right)(150) = 142.05 \ \frac{\text{kips}}{\text{pier}}$$

$$M_x = (142.05)(13.75) + (48.27)(19.13) = 2876.60 \ \frac{\text{kip ft}}{\text{pier}}$$

(2) Reference el 1216.2

$$F_y = F_H = 48.27 \ \frac{\text{kips}}{\text{pier}}$$

$$F_z \quad F_V = 142.05 \ \frac{\text{kips}}{\text{pier}}$$

$$M_x = (142.05)(13.75) + (48.27)(20.8) = 2957.21 \ \frac{\text{kip ft}}{\text{pier}}$$

90. The conventional stability analysis is accurate enough to check and determine that the posttensioning does not overstress the piles. The results are presented in Tables 20-23.

91. With the posttensioning, adequate resistance to shear at the top of the piling and the stresses in the piles will be adequate. This will make the piling system at Pine River Dam adequate in stability.

PART VI: CONCLUSIONS AND RECOMMENDATIONS

Foundation

92. The foundation material has reliable in situ supporting capabilities. The pilings have been continuously submerged and therefore should be nondeteriorated and adequate. During the drilling program for the Upper Mississippi Headwater Structures, pieces of planks, beams, and piling were obtained at various locations that supported this conclusion.

93. The piling layout is not adequate because the shear stresses at the top of the piles and stresses in some piles are excessive. It is recommended that the dam monoliths be posttensioned to the foundation as explained in Part V and in Figure 65. The design of the anchor length for the slant-hole soil anchor system and the system itself is somewhat dependent on the anchors used and is left to be designed by the contractor.

Concrete

94. The concrete is of good quality with the only deterioration caused by surface freezing and thawing. Core PR-P3 indicates that the concrete in the piers does vary in quality and any poor concrete found during posttensioning should be evaluated in relation to its performance after posttensioning. It is recommended that within the next 5 years an acrylic-polymer coating as discussed in Part IV be investigated and, if adequate, be used to rehabilitate the deteriorated surface concrete. It is necessary to rehabilitate the surface concrete in order to stop water from entering cracks and accelerating the deterioration of the interior concrete as the freezing and thawing process continues at Pine River Dam. If the deterioration is allowed to continue until major replacement of the concrete is required, the rehabilitation could be very expensive.

95. After the remedial measures and surface concrete rehabilitation, it is expected that this structure will be adequate for many more years of service.

REFERENCES

Baguelin, F., Kiziquel, J. F., and Shields, D. H. 1978. "The Pressuremeter and Foundation Engineering," 1st ed., Trans Tech Publications.

Hrennikoff, A. 1950. "Analysis of Pile Foundations with Batter Piles," Transactions, American Society of Civil Engineers, Vol 76, No. 1, Paper No. 2401, pp 123-126.

Martin, Deborah K., Jones, H. Wayne, Radhakrishnan, N. 1980 (Jun). "Documentation for LMVDPILE Program," Technical Report K-80-3, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

Saul, W. E. 1968. "Static and Dynamic Analysis of Pile Foundations," Journal, Structural Division, American Society of Civil Engineers, Vol 94, No. SM5, pp 1071-1100.

U. S. Army Engineer District, St. Paul, CE. 1973. "Reservoirs at Headwaters, Mississippi River, Minnesota, Gull Lake Dam," Periodic Inspection Report No. 1, St. Paul, Minn.

U. S. Army Engineer District, St. Paul, CE. 1977. "Mississippi River, Headwaters Reservoirs, Master Plan for Public Use Development and Resource Management," St. Paul, Minn.

Southern Pine Association, "Modern Timber Engineering," Scofield and O'Brien, 1954.

Table 1

General Resrvoir Data

Location in miles above Ohio River	1038.3
Located on river	Pine
Drainage area (square miles)	562
Original operating limits	
Stage	3.18*-20.38 ft
Storage in 1000 acre-feet	179
Present operating limits	
Stage	10.88 to 20.38 ft
Storage in 1000 acre-feet	80.6
Ordinary operating limits	
Stage	12.88-15.88 ft
Storage in 1000 acre-feet	41
Flowage rights to stage	24.38
Maximum stage of record (1916)	20.12
Number of times upper operating limit exceeded	0
Number of times flowage limit exceeded	0
Maximum stage in 1950	16.97
Maximum discharge of record	2250 sec-ft
and year	1896
Elevation of gage zero: U.S.E. datum	1218.20
Elevation of gage zero: msl (1929 adj.)	1216.32
Year of first operation	1886
Normal spring stage drawdown	12.88 ft
Normal summer range	14.63-15.13 ft
Desirable bridge clearance, 9 ft above reservoir stage of	16.88 ft

* All stages in this table are referred to msl, 1929 adj.

Table 2

Pertinent Dam Data

Dam	
Type	Earth fill with timber diaphragm core filled with puddled clay
Crest height	24.38* ft
Length	1265 ft
Height (maximum)	25.98 ft
Freeboard above maximum stage	4.0 ft
Control Structure	
Type	Reinforced concrete
Sill height	2.21 ft
Net length of spillway	78 ft
Height of piers	21.38 ft
Sluiceways	
Width	6 ft
Number of bays	13
Number of sluice gates (54 by 60 in.)	11
Number of stop log sections	2
Height of stop logs at normal pool	14.88 ft
Discharge channel capacity	Not determined
Spillway Apron	
Type	Concrete
Length	86 ft
Width (between abutments)	150 ft
Floor height	2.21 ft
Bridge Over Control Structure	
Height of roadway**	22.38 ft
Roadway width (for pedestrian use only)	8 ft

* All heights or stages are referred to msl, 1929 adj.

** No longer a public roadway. The highway was relocated; a new highway bridge crosses Pine River about 450 ft downstream from dam.

Table 3

Pressuremeter Probe Location

Below Pier Bottom

Hole	Test	Probe Location, ft
PR-P3	Test 1	4.22
	Test 2	Test Void
	Test 3	16.12
PR-P6	Test 1	7.92
	Test 2	12.52
	Test 3	16.92
PR-P9	No Pressuremeter Data.	

Table 4

Split-Spoon Test Results, Pine River Dam

Hole	Test	Depth from Bottom of Concrete, ft	Number of Blows
PR-P3	1	0.0 - 1.0	1
		1.0 - 1.5	2
		1.5 - 1.9	1
	2	5.3 - 5.8	4
		5.8 - 6.3	14
		6.3 - 6.8	15
PR-P6	1	0.6 - 1.1	6
		1.1 - 1.6	5
		1.6 - 2.0	50
	2	9.5 - 10.0	9
		10.0 - 10.5	18
		10.5 - 11.0	24
	3	13.5 - 13.9	50
PR-P9	1	0.5 - 1.0	11
		1.0 - 1.5	9
		1.5 - 2.0	6
	2	15.2 - 15.7	41
		15.7 - 16.0	50

Table 5

Unconfined Compressive Concrete Strengths

Core Hole	Specimen	Unconfined Compressive Strength, psi
PR-P3	PR-P3M	5800
	PR-P3B	5800
PR-3A	PR-P3AT	6000
	PR-P3AM	3800
	PR-P3AB	4000
PR-P6	PR-P6T	7000
	PR-P6M	5300
	PR-P6B	5900
PR-P9	PR-P9T	7080
	PR-P9M	5500
	PR-P9B	5600

NOTE: Average compressive value ≈ 5600.

Table 6

Patching Material for Cracks,
Spalled Joints, and Holes

Material	Parts by Mass
Cement	100
Water	≈ 18 (adjust as needed)
Acrylic Polymer	27
Fine Sand (Passing No. 30 Sieve)	150

Table 7

Overlay Material for
Surface Concrete Rehabilitation

Material	Parts by Mass
Cement	100
Water	≈ 20 (adjust as needed)
Acrylic Polymer	30

Table 8

Forces at Top of 8 Piles by Conventional Analysis

(reference el 1215.99, tailwater el 1215.12)

Case Loading	Horizontal Load F_H kips	Number of Piles	Horizontal Load per Pile kips	F_z kips	e (from Moment Center) ft	Moment About Center of Gravity of Pile Group $F_z(9.47 - e)$ kip-ft	Moment of Inertia of Pile Group, ft^4	Maximum Compressive Force Per Pile kips	Minimum Tensile Force Per Pile, kips
Normal operation	85.84	8	10.7	276.64	11.05	-437	308	46	0
Normal operation with truck loading (H15-44)	85.84	8	10.7	300.64	11.21	-523	308	51	0
Normal operation with earthquake	96.67	8	12.1	276.64	10.68	-335	308	43	0
Normal operation with ice	145.87	8	18.2	276.64	7.88	440	308	46	0
High-water condition	135.03	8	16.9	251.70	9.34	33	308	32	0

Table 9

Forces at Top of 8 Piles by Conventional Analysis

(reference el 1214.32, tailwater el 1215.12)

Case Loading	Horizontal Load F_H kips	Number of Piles	Horizontal Load per Pile kips	F_z kips	e (from Moment Center) ft	Moment About Center of Gravity of Pile Group $F_z(9.47 - e)$ kip-ft	Moment of Inertia of Pile Group, ft^4	Maximum Compressive Force Per Pile kips	Minimum Tensile Force Per Pile, kips
Normal operation	105.60	8	13.2	308.57	10.49	-315	308	47	0
Normal operation with truck loading (H15-44)	105.60	8	13.2	332.57	10.67	-399	308	52	0
Normal operation with earthquake	117.78	8	14.7	308.57	10.09	-191	308	43	0
Normal operation with ice	165.60	8	20.7	308.57	7.32	663	308	55	0
High-water condition	157.06	8	19.6	278.29	8.59	245	308	41	0

Table 10

Forces at Top of 8 Piles by Conventional Analysis

(reference at 1215.99, tailwater el 1220.12)

Case Loading	Horizontal Load F_H kips	Number of Piles	Horizontal Load per Pile kips	F_z kips	e (from Moment Center) ft	Moment About Center of Gravity of Pile Group $F_z(9.47 - e)$ kip-ft	Moment of Inertia of Pile Group, ft^4	Maximum Compressive Force Per Pile kips	Minimum Tensile Force Per Pile, kips
Normal operation	82.00	8	10.3	266.26	11.19	-458	308	45	0
Normal operation with truck loading (H15-44)	82.00	8	10.3	290.26	11.34	-543	308	50	0
Normal operation with earthquake	92.83	8	11.6	266.66	10.80	-355	308	42	0
Normal operation with ice	142.00	8	17.8	266.26	7.89	421	308	44	0
High-water condition	Not most critical case.								

Table 11

Forces at Top of 8 Piles by Conventional Analysis

(reference el 1214.32, tailwater el 1220.12)

Case Loading	Horizontal Load F_H kips	Number of Piles	Horizontal Load per Pile kips	F_z kips	e (from Moment Center) ft	Moment About Center of Gravity of Pile Group $F_z(9.47 - e)$ kip-ft	Moment of Inertia of Pile Group, ft^4	Maximum Compressive Force Per Pile kips	Minimum Tensile Force Per Pile, kips
Normal operation	98.27	8	12.3	284.88	10.62	-328	308	44	0
Normal operation with truck loading (H15-44)	98.27	8	12.3	308.88	10.81	-414	308	49	0
Normal operation with earthquake	110.45	8	13.8	284.88	10.19	-205	308	41	0
Normal operation with ice	158.27	8	19.8	284.88	7.19	650	308	52	0
High-water condition	Not most critical case.								

Table 12

Forces at Top of 14 Piles by Conventional Analysis

(reference el 1215.99, tailwater el 1215.12)

Case Loading	Horizontal Load F_H kips	Number of Piles	Horizontal Load per Pile kips	F_z kips	e (from Moment Center) ft	Moment About Center of Gravity of Pile Group $F_z(9.52 - e)$ kip-ft	Moment of Inertia of Pile Group, ft^4	Maximum Compressive Force Per Pile kips	Minimum Tensile Force Per Pile, kips
Normal operation	85.84	14	6.1	276.64	11.05	-423	364	29	0
Normal operation with truck loading (H15-44)	85.84	14	6.1	300.64	11.21	-508	364	32	0
Normal operat. n with earthquake	96.67	14	6.9	276.64	10.68	-321	364	27	0
Normal operation with ice	145.87	14	10.4	276.64	7.88	454	364	29	0
High-water condition	135.03	14	9.6	251.70	9.34	45	364	19	0

Table 13

Forces at Top of 14 Piles by Conventional Analysis

(reference el 1214.32, tailwater el 1215.12)

Case Loading	Horizontal Load F_H kips	Number of Piles	Horizontal Load per Pile kips	F_z kips	e (from Moment Center) ft	Moment About Center of Gravity of Pile Group $F_z(9.52 - e)$ kip-ft	Moment of Inertia of Pile Group, ft^4	Maximum Compressive Force Per Pile kips	Minimum Tensile Force Per Pile, kips
Normal operation	105.60	14	7.5	308.57	10.49	-299	364	28	0
Normal operation with truck loading (H15-44)	105.60	14	7.5	332.57	10.67	-382	364	32	0
Normal operation with earthquake	117.78	14	8.4	308.57	10.09	-176	364	26	0
Normal operation with ice	165.60	14	11.8	308.57	7.32	679	364	37	0
High-water condition	157.06	14	11.2	278.29	8.59	259	364	25	0

Table 14

Forces at Top of 14 Piles by Conventional Analysis

(reference el 1215.99, tailwater el 1220.2)

Case Loading	Horizontal Load F_H kips	Number of Piles	Horizontal Load per Pile kips	F_z kips	e (from Moment Center) ft	Moment About Center of Gravity of Pile Group $F_z(9.52 - e)$ kip-ft	Moment of Inertia of Pile Group, ft^4	Maximum Compressive Force Per Pile kips	Minimum Tensile Force Per Pile, kips
Normal operation	82.00	14	5.9	266.26	11.19	-445	364	28	0
Normal operation with truck loading (H15-44)	82.00	14	5.9	290.26	11.34	-528	364	32	0
Normal operation with earthquake	92.83	14	6.6	266.66	10.80	-341	364	26	0
Normal operation with ice	142.00	14	10.1	266.66	7.89	+434	364	28	0
High-water condition	Not most critical case.								

Table 15

Forces at Top of 14 Piles by Conventional Analysis

(reference el 1214.32, tailwater el 1220.12)

Case Loading	Horizontal Load F_H kips	Number of Piles	Horizontal Load per Pile kips	F_z kips	e (from Moment Center) ft	Moment About Center of Gravity of Pile Group $F_z(9.52 - e)$ kip-ft	Moment of Inertia of Pile Group, ft^4	Maximum Compressive Force Per Pile kips	Minimum Tensile Force Per Pile, kips
Normal operation	98.27	14	7.0	284.88	10.62	-423	364	36	0
Normal operation with truck loading (H15-44)	98.27	14	7.0	308.88	10.81	-398	364	31	0
Normal operation with earthquake	110.45	14	7.9	284.88	10.19	-191	364	24	0
Normal operation with ice	158.27	14	11.3	284.88	7.19	664	364	35	0
High-water condition	Not most critical case.								

Table 16

Pine River Dam, Pier Supported by Group of 8 Piles

(bottom of pier at el 1215.99)

Load Case	Elevation of Headwater, ft	Elevation of Tailwater, ft	Lateral Force at Top of Pile U1, kips	Lateral Deflection At Top of Pile U1, in.	Maximum		Minimum	
					Axial Force At Top of Pile F3, kips	Axial Deflection At Top of Pile U3, in.	Axial Force At Top of Pile F3, kips	Axial Deflection At Top of Pile U3, kips
Normal operation	1231.12	1215.12	10.73	0.0512	45.63	0.0559	23.62	0.0289
Normal operation with truck loading (H15-44)	1231.12	1215.12	10.73	0.0512	50.81	0.0623	24.46	0.0300
Normal operation with earthquake	1231.12	1215.12	12.08	0.0576	43.05	0.0527	26.18	0.0321
Normal operation with ice	1231.12	1215.12	18.23	0.0869	45.60	0.0559	23.47	0.0288
High-water condition	1235.12	1219.12	16.88	0.0805	32.28	0.0395	30.64	0.0375
Normal operation	1231.12	1220.12	10.25	0.0489	44.86	0.0550	21.79	0.0267
Normal operation with truck loading (H15-44)	1231.12	1220.12	10.25	0.0489	50.01	0.0613	22.67	0.0278
Normal operation with earthquake	1231.12	1220.12	11.60	0.0553	42.24	0.0518	24.40	0.0299
Normal operation with ice	1231.12	1220.12	17.75	0.0846	43.82	0.0537	22.66	0.0278

Table 17

Pine River Dam, Pier Supported by Group of 8 Piles

(bottom of pier at el 1214.32)

Load Case	Elevation of Headwater, ft	Elevation of Tailwater, ft	Lateral Force at Top of Pile U1, kips	Lateral Deflection At Top of Pile U1, in.	Maximum: Axial Force At Top of Pile F3, kips	Maximum: Axial Deflection At Top of Pile U3, in.	Minimum: Axial Force At Top of Pile F3, kips	Minimum: Axial Deflection At Top of Pile U3, kips
Normal operation	1231.12	1215.12	13.20	0.0630	46.53	0.0570	30.67	0.0376
Normal operation with truck loading (H15-44)	1231.12	1215.12	13.20	0.0630	51.67	0.0633	31.56	0.0387
Normal operation with earthquake	1231.12	1215.12	14.72	0.0702	43.42	0.0532	33.77	0.0414
Normal operation with ice	1231.12	1215.12	20.70	0.0987	55.19	0.0676	21.82	0.0267
High-water condition	1235.12	1219.12	19.63	0.0936	40.92	0.0501	28.61	0.0351
Normal operation	1231.12	1220.12	12.28	0.0586	43.90	0.0538	27.39	0.0336
Normal operation with truck loading (H15-44)	1231.12	1220.12	12.28	0.0586	49.07	0.0601	28.24	0.0346
Normal operation with earthquake	1231.12	1220.12	13.81	0.0658	40.80	0.0500	30.46	0.0373
Normal operation with ice	1231.12	1220.12	19.78	0.0943	51.88	0.0636	19.21	0.0235

Table 18

Pine River Dam, Pier Supported by Group of 14 Piles

(bottom of pier at el 1215.99)

Load Case	Elevation of Headwater, ft	Elevation of Tailwater, ft	Lateral Force at Top of Pile U1, kips	Lateral Deflection At Top of Pile U1, in.	Maximum Axial Force At Top of Pile F3, kips	Maximum Axial Deflection At Top of Pile U3, in.	Minimum Axial Force At Top of Pile F3, kips	Minimum Axial Deflection At Top of Pile U3, kips
Normal operation	1231.12	1215.12	6.13	0.0396	28.76	0.0352	10.72	0.0131
Normal operation with truck loading (H15-44)	1231.12	1215.12	6.13	0.0396	32.27	0.0395	10.63	0.0130
Normal operation with earthquake	1231.12	1215.12	6.90	0.0446	26.58	0.0326	12.90	0.0158
Normal operation with ice	1231.12	1215.12	10.42	0.0673	29.42	0.0361	10.14	0.0124
High-water condition	1235.12	1219.12	9.64	0.0624	18.93	0.0232	17.03	0.0209
Normal operation	1231.12	1220.12	5.86	0.0379	28.47	0.0349	9.52	0.0117
Normal operation with truck loading (H15-44)	1231.12	1220.12	5.86	0.0379	31.96	0.0392	9.46	0.0116
Normal operation with earthquake	1231.12	1220.12	6.63	0.0429	26.26	0.0322	11.74	0.0144
Normal operation with ice	1231.12	1220.12	10.14	0.0656	28.26	0.0346	9.82	0.0120

Table 19

Pine River Dam, Pier Supported by Group of 14 Piles

(bottom of pier at el 1214.32)

Load Case	Elevation of Headwater, ft	Elevation of Tailwater, ft	Lateral Force at Top of Pile U1, kips	Lateral Deflection At Top of Pile U1, in.	Maximum		Minimum	
					Axial Force At Top of Pile F3, kips	Axial Deflection At Top of Pile U3, in.	Axial Force At Top of Pile F3, kips	Axial Deflection At Top of Pile U3, kips
Normal operation	1231.12	1215.12	7.54	0.0488	28.41	0.0348	15.64	0.0192
Normal operation with truck loading (H15-44)	1231.12	1215.12	7.54	0.0488	31.89	0.0391	15.58	0.0191
Normal operation with earthquake	1231.12	1215.12	8.41	0.0544	25.79	0.0316	18.28	0.0224
Normal operation with ice	1231.12	1215.12	11.83	0.0765	36.50	0.0447	7.65	0.0094
High-water condition	1235.12	1219.12	11.22	0.0725	25.38	0.0311	14.40	0.0176
Normal operation	1231.12	1220.12	7.02	0.0454	27.01	0.0331	13.65	0.0167
Normal operation with truck loading (H15 44)	1231.12	1220.12	7.02	0.0454	30.53	0.0374	13.55	0.0166
Normal operation with earthquake	1231.12	1220.12	7.89	0.0510	24.41	0.0299	16.27	0.0199
Normal operation with ice	1231.12	1220.12	11.30	0.0731	34.49	0.0423	6.27	0.0077

Table 20

Forces at Top of 8 Piles by Conventional Analysis After Posttensioning

(reference el 1215.99, tailwater el 1215.12)

Case Loading	Horizontal Load F_H kips	Number of Piles	Horizontal Load per Pile kips	F_z kips	e (from Moment Center) ft	Moment About Center of Gravity of Pile Group $F_z(9.47 - e)$ kip-ft	Moment of Inertia of Pile Group, ft^4	Maximum Compressive Force Per Pile kips	Minimum Tensile Force Per Pile, kips
Normal operation	38	8	4.8	419	14.17	-1969	242	102	0
Normal operation with truck loading (H15-44)	38	8	4.8	443	14.11	-2056	242	107	0
Normal operation with earthquake	48	8	6.0	419	13.92	-1865	242	100	0
Normal operation with ice	98	8	12.3	419	12.07	-1089	242	80	0
High-water condition	87	8	10.9	394	13.28	-1501	242	87	0

Table 21

Forces at Top of 8 Piles by *Conventional Analysis* After Posttensioning

(reference el 1214.32, tailwater el 1215.12)

Case Loading	Horizontal Load F_H kips	Number of Piles	Horizontal Load per Pile kips	F_z kips	e (from Moment Center) ft	Moment About Center of Gravity of Pile Group $F_z(9.47 - e)$ kip-ft	Moment of Inertia of Pile Group, ft^4	Maximum Compressive Force Per Pile kips	Minimum Tensile Force Per Pile, kips
Normal operation	57	8	7.1	451	13.74	-1926	242	105	0
Normal operation with truck loading (H15-44)	57	8	7.1	475	13.71	-2014	242	110	0
Normal operation with earthquake	70	8	8.8	451	13.47	-1804	242	102	0
Normal operation with ice	117	8	14.6	451	11.57	-947	242	80	0
High-water condition	109	8	13.6	420	12.72	-1365	242	87	0

Table 22

Forces at Top of 8 Piles by Conventional Analysis After Posttensioning

(reference el 1215.99, tailwater el 1220.12)

Case Loading	Horizontal Load F_H kips	Number of Piles	Horizontal Load per Pile kips	F_z kips	e (from Moment Center) ft	Moment About Center of Gravity of Pile Group $F_z(9.47 - e)$ kip-ft	Moment of Inertia of Pile Group, ft^4	Maximum Compressive Force Per Pile kips	Minimum Tensile Force Per Pile, kips
Normal operation	34	8	4.3	408	14.34	-1987	242	101	0
Normal operation with truck loading (H15-44)	34	8	4.3	432	14.27	-2074	242	107	0
Normal operation with earthquake	45	8	5.6	408	14.09	-1885	242	99	0
Normal operation with ice	93	8	11.6	408	12.19	-1110	242	79	0
High-water condition	Not most critical case.								

Table 23

Forces at Top of 8 Piles by Conventional Analysis After Posttensioning

(reference el 1214.32, tailwater el, 1220.2)

Case Loading	Horizontal Load F_H kips	Number of Piles	Horizontal Load per Pile kips	F_z kips	e (from Moment Center) ft	Moment About Center of Gravity of Pile Group $F_z(9.47 - e)$ kip-ft	Moment of Inertia of Pile Group, ft^4	Maximum Compressive Force Per Pile kips	Minimum Tensile Force Per Pile, kips
Normal operation	50	8	6.3	427	14.02	-1943	242	103	0
Normal operation with truck loading (H15-44)	50	8	6.3	451	13.96	-2025	242	108	0
Normal operation with earthquake	62	8	7.8	427	13.73	-1819	242	99	0
Normal operation with ice	110	8	13.8	427	11.73	-965	242	78	0
High-water condition	Not most critical case.								

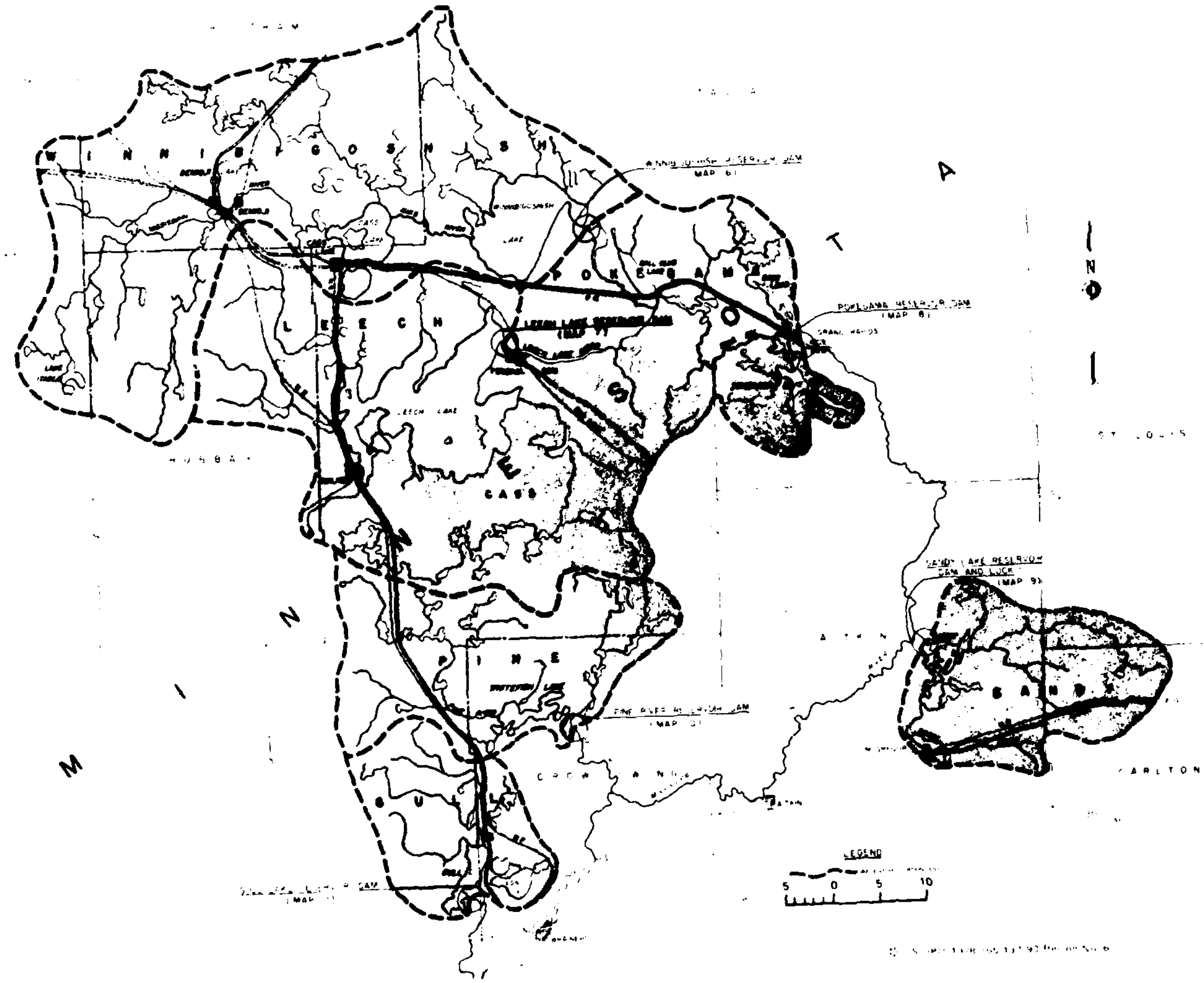


Figure 1. Project map of Mississippi River headwaters reservoirs

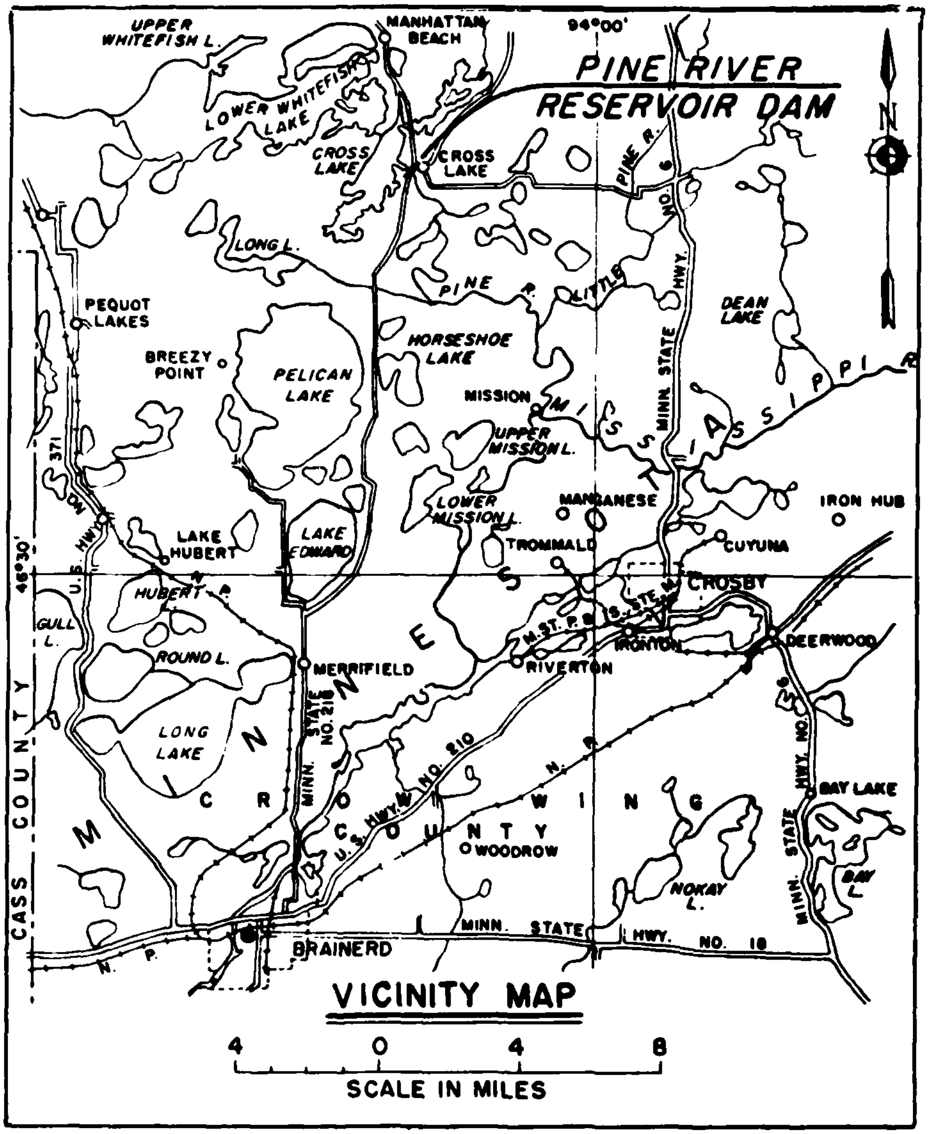


Figure 2. Vicinity map of Pine River area

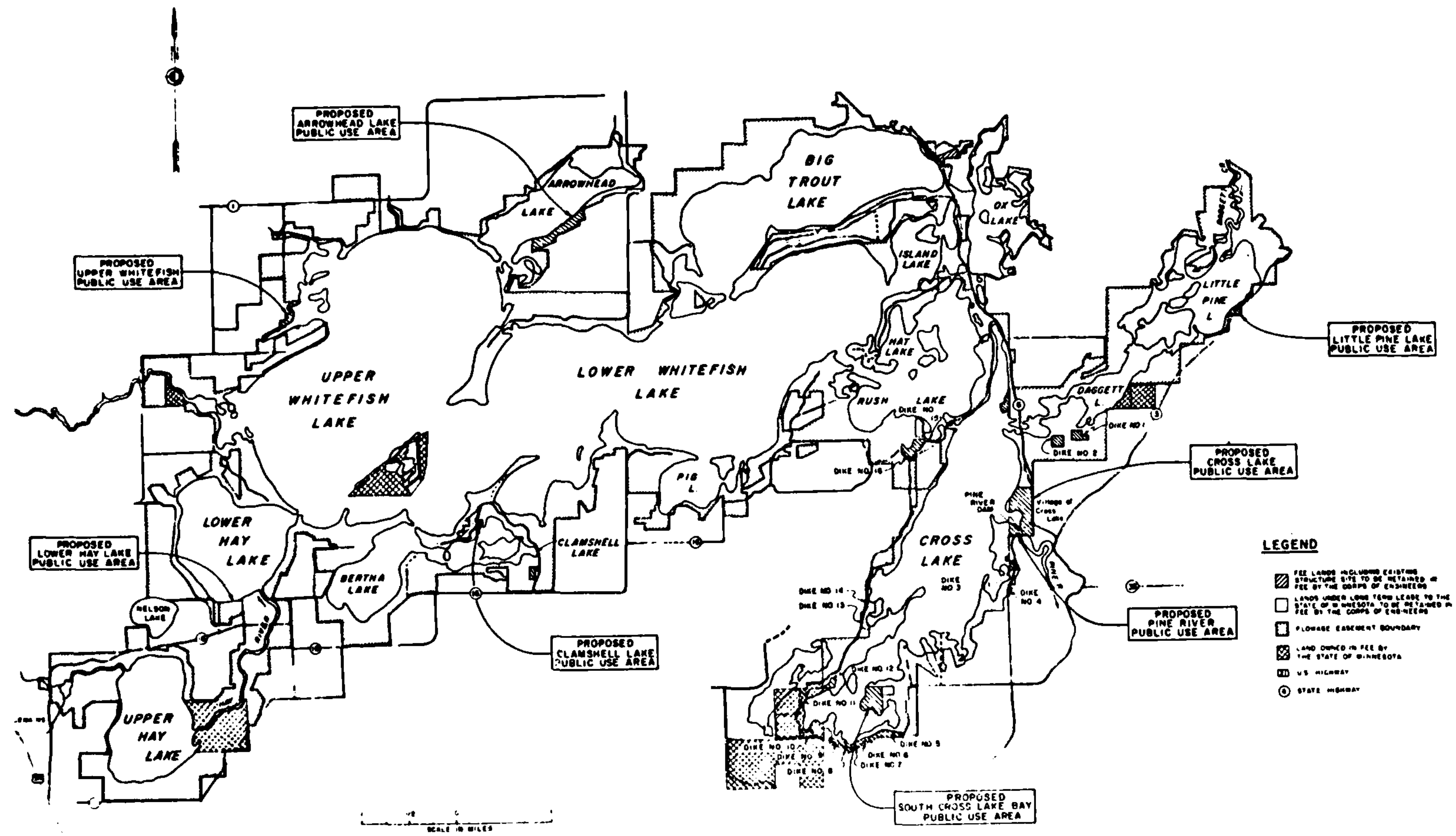
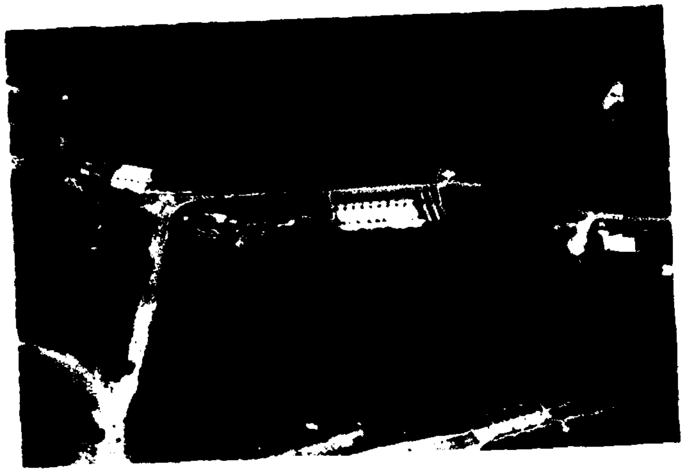
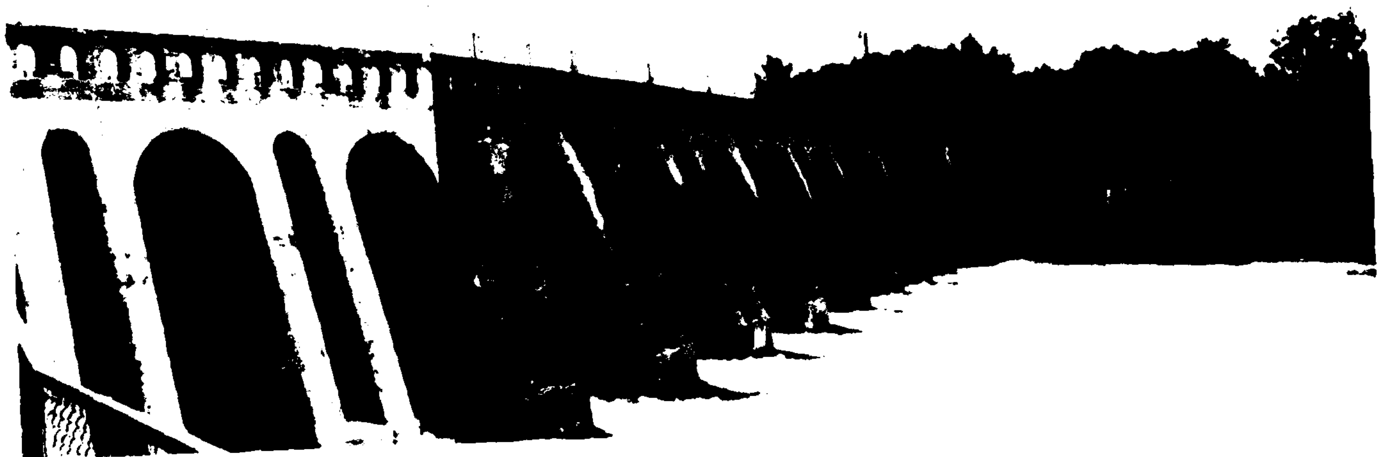


Figure 3. Master plan for reservoir development, Pine River Reservoir



a. Aerial site photo



b. Panoramic view from downstream right abutment

Figure 4. Control structure at Pine River Dam

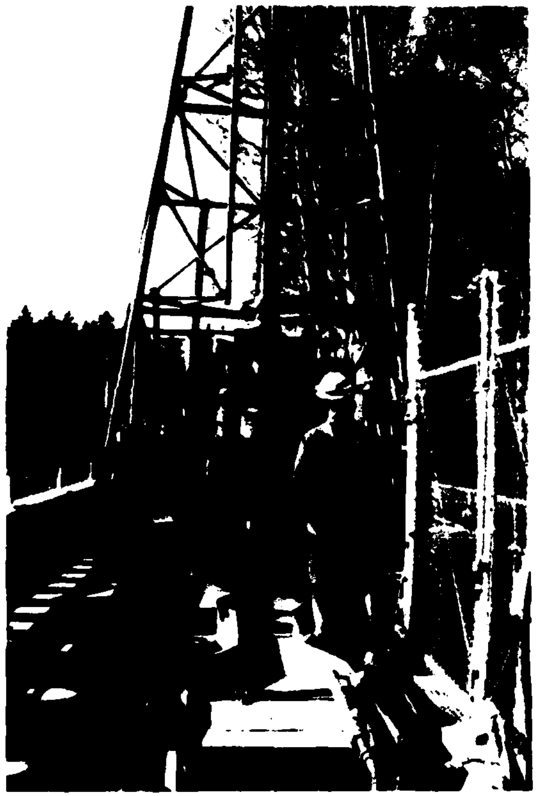


Figure 5. Schematic plan view presenting location of core holes, piers PR-P3, PR-P6, and PR-P9

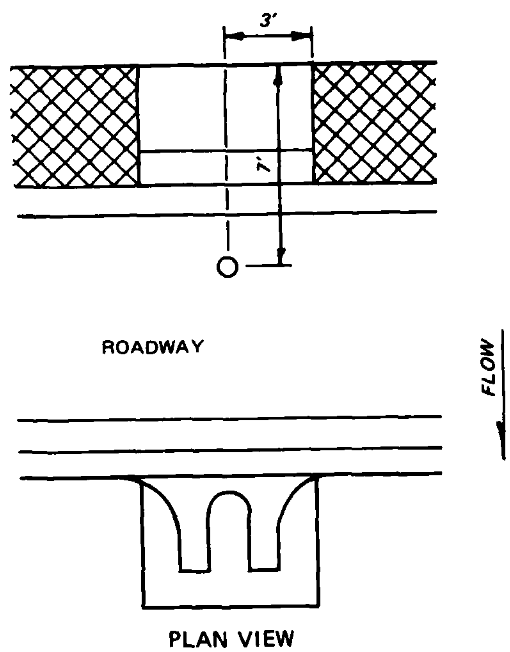
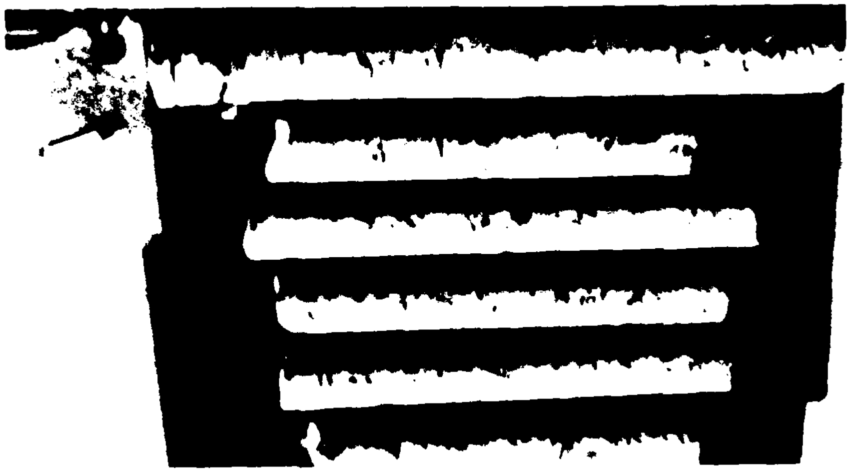
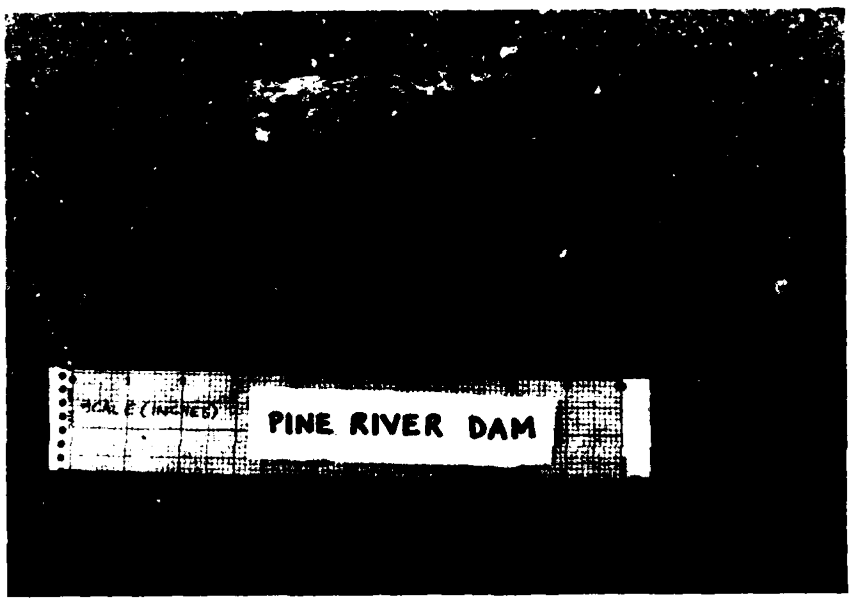


Figure 6. Typical drilling setup



a. Concrete core



b. Core and cut section

Figure 7. Typical views of concrete core

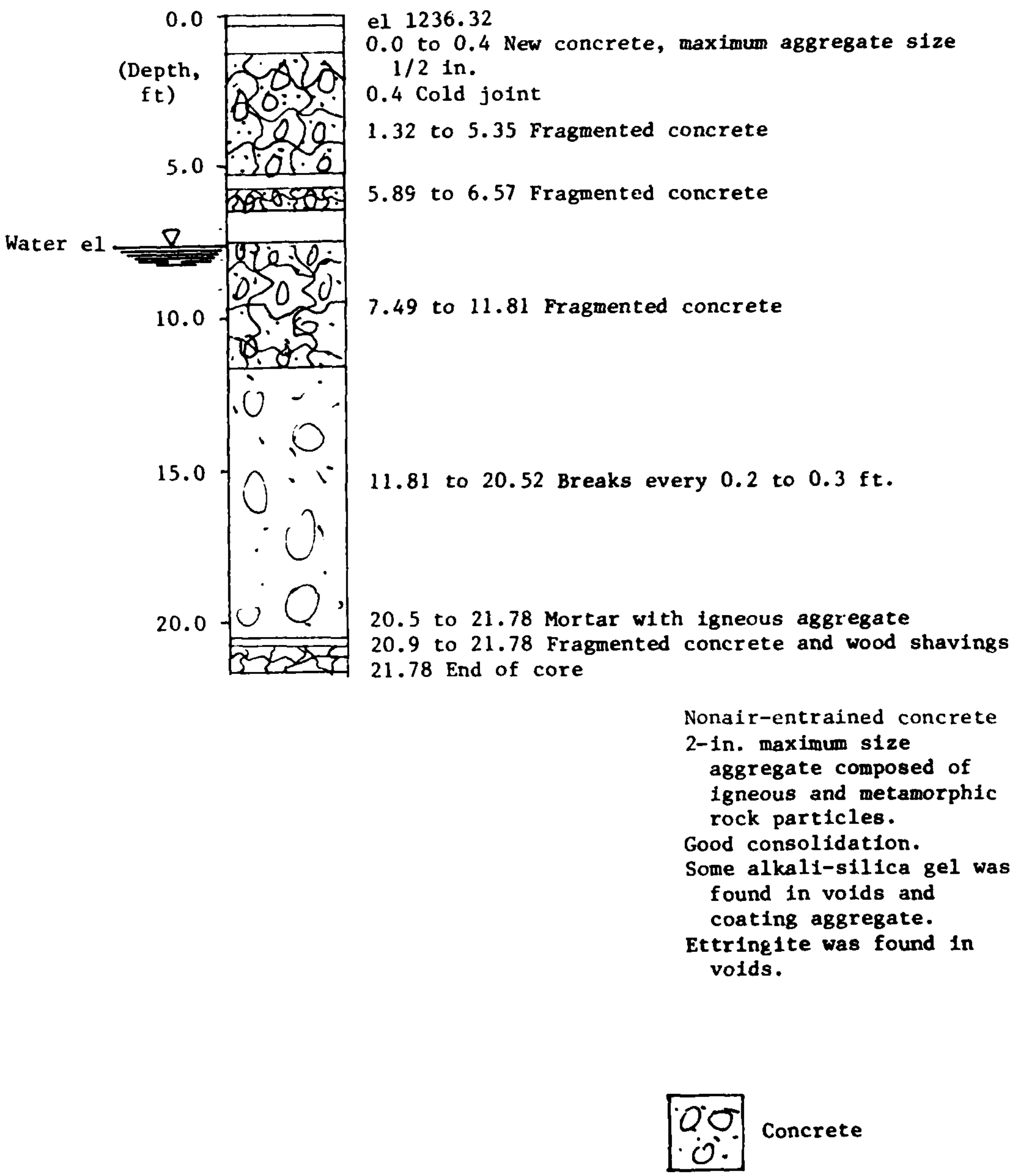
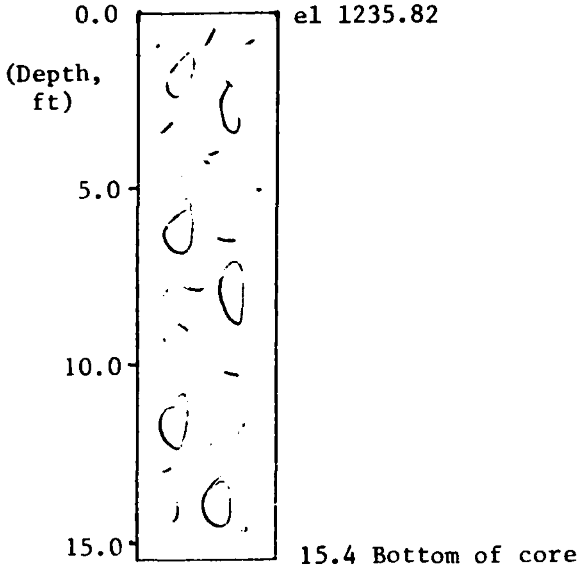


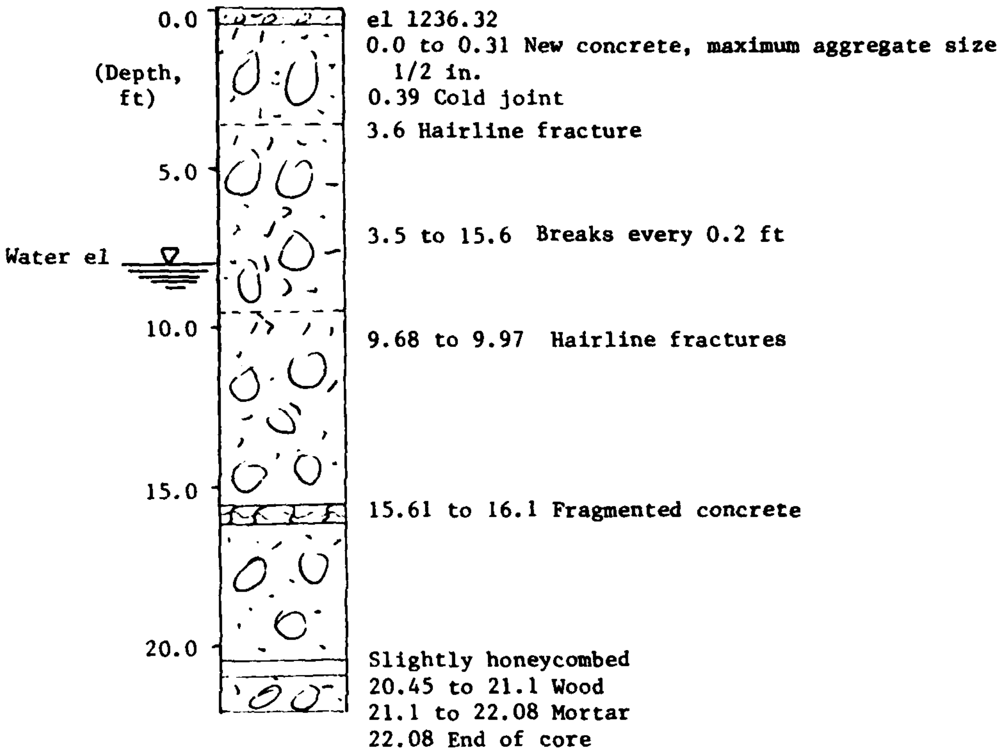
Figure 8. Vertical NX concrete core, PR-P3, Pine River Dam



Nonair-entrained concrete.
2-in. maximum size aggregate composed of igneous and metamorphic rock particles.
Good consolidation.
Some alkali-silica gel was found in voids and coating aggregate.
Ettringite was found in voids.
Although not shown, breaks due to drilling occur every 2 to 3 ft.


Concrete

Figure 9. Vertical 4-in.-diam concrete core, PR-P3A, Pine River Dam



Nonair-entrained concrete.
2-in. maximum size aggregate composed of igneous and metamorphic rock particles.
Generally good consolidation.
Some alkali-silica gel was found in voids and coating aggregate.
Ettringite was found in voids.


Concrete

Figure 10. Vertical NX concrete core, PR-P6, Pine River Dam

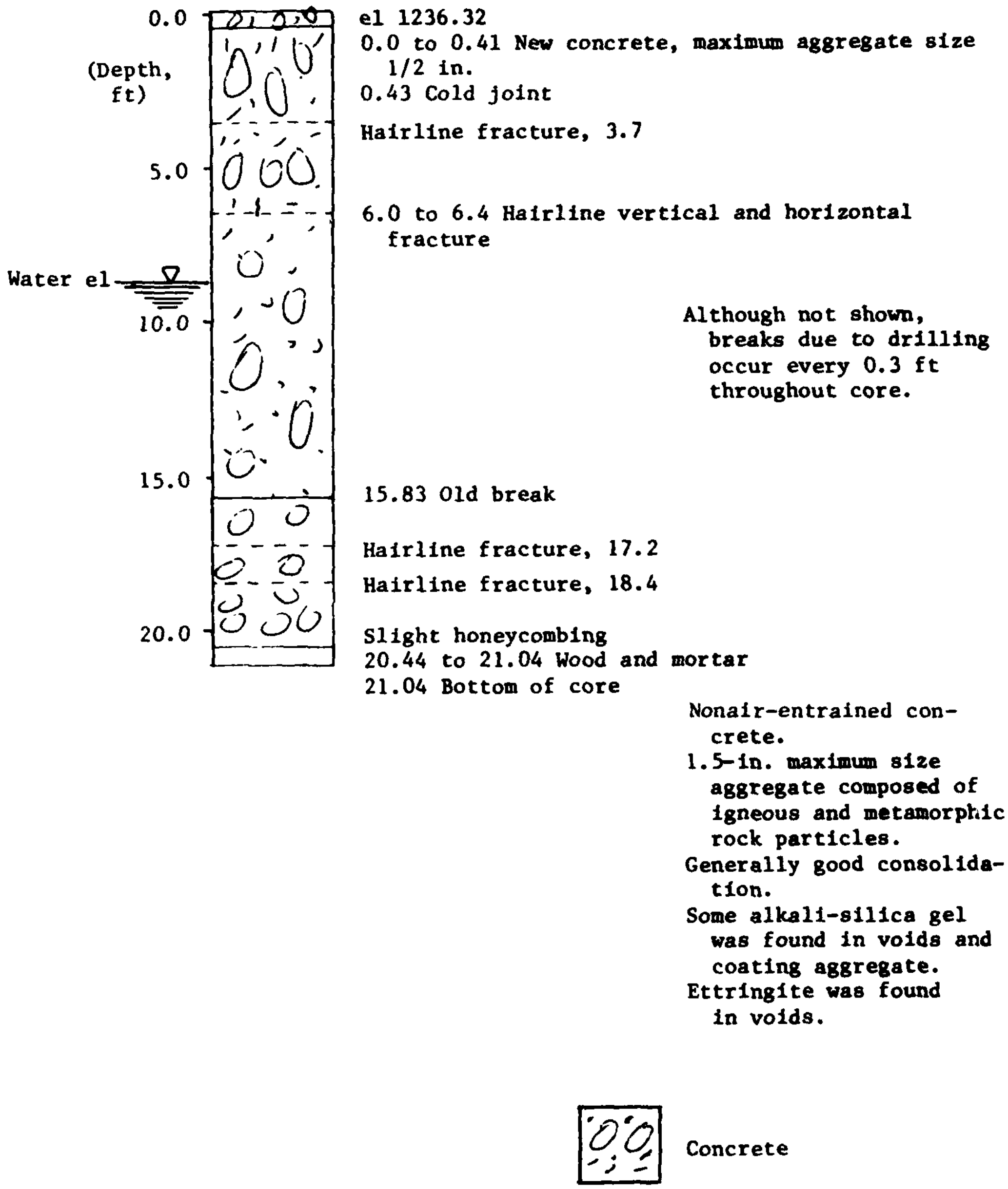


Figure 11. Vertical NX concrete core, PR-P9, Pine River Dam

Figure 12. In this SEM micrograph (magnified 25 times) the area outlined is alkali-silica reaction gel in an air void that was adjacent to a piece of granitic coarse aggregate. The gel has dried and cracked

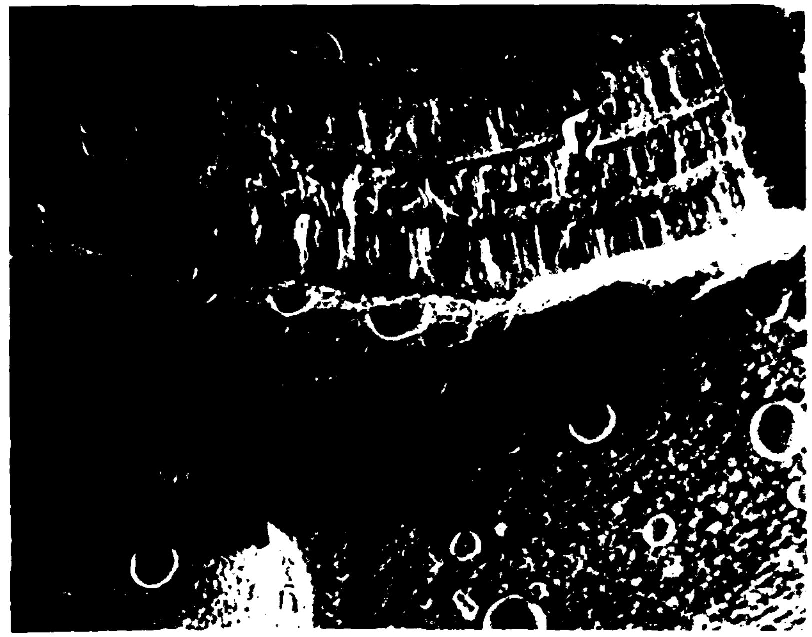


Figure 13. Enlarged view of gel from upper portion of previous SEM micrograph (magnified 250 times). The layered structure of some of the gel can be seen in the fractured surface of the ridge extending across the top of the micrograph

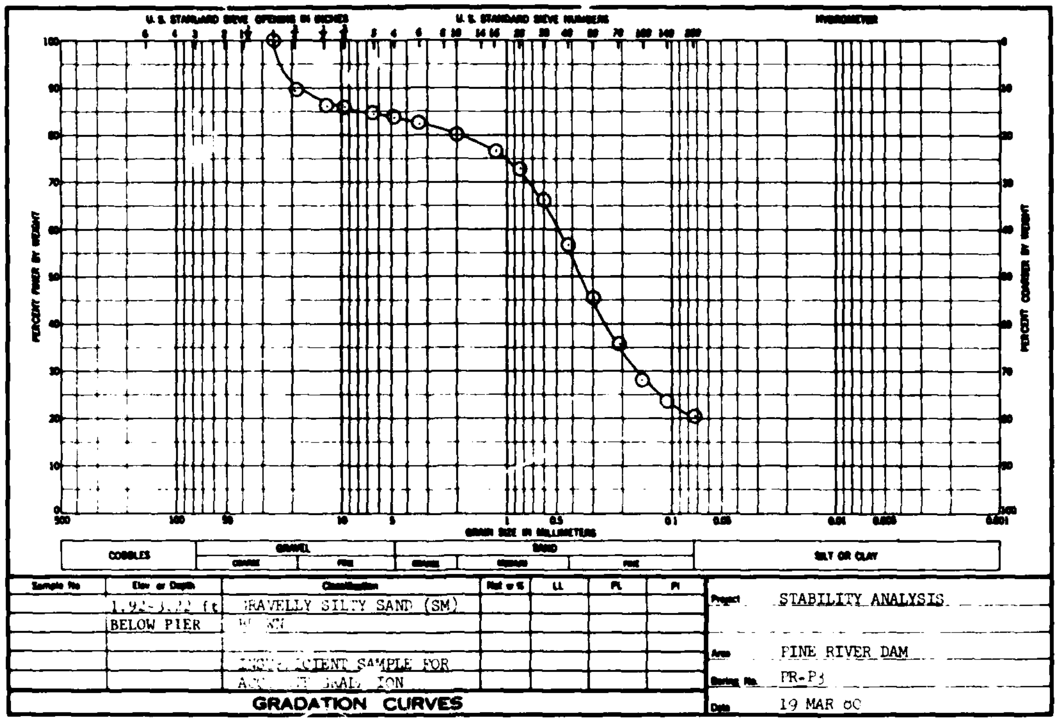


Figure 14. Foundation soil sieve analysis and classification, 1.92-3.22 ft

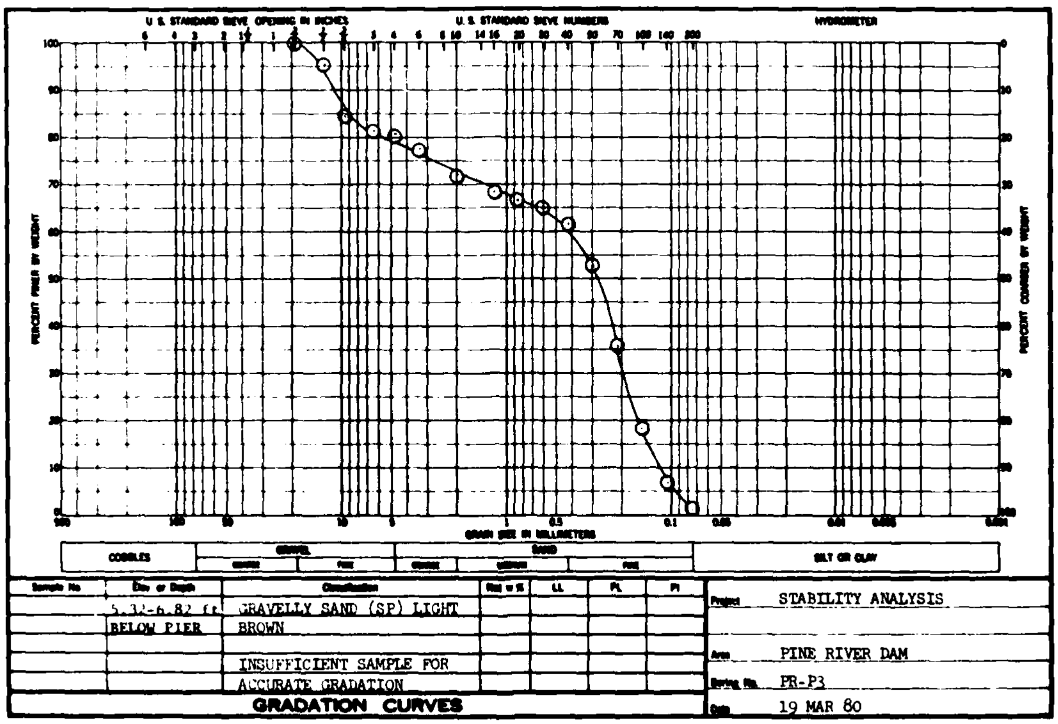


Figure 15. Foundation soil sieve analysis and classification, 5.32-6.82 ft

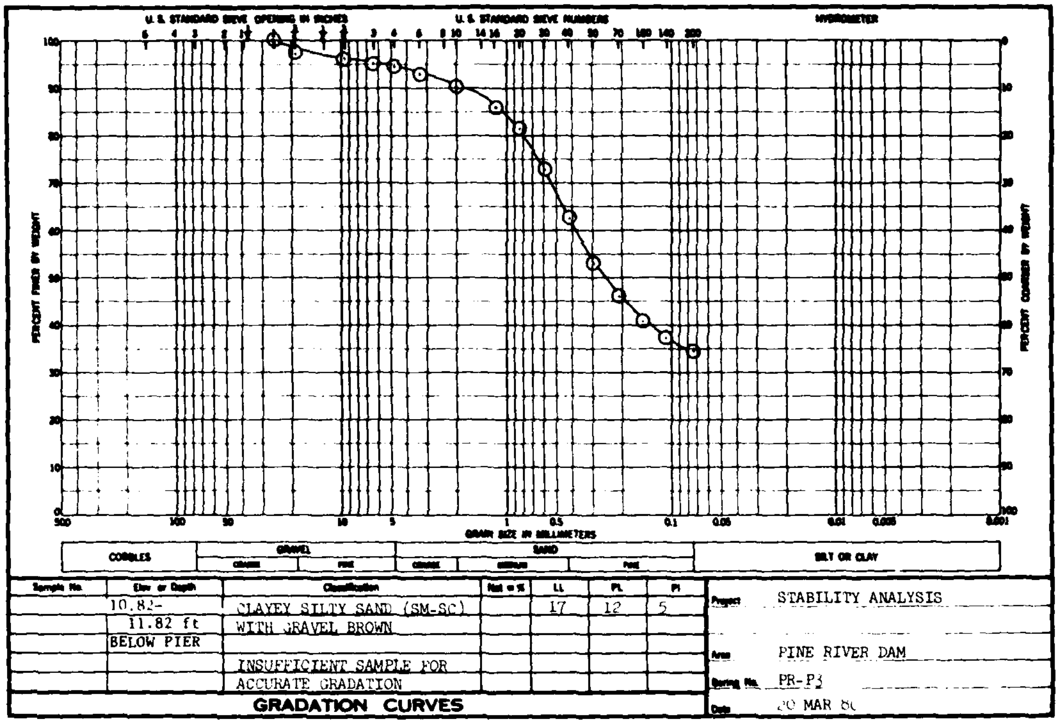


Figure 16. Foundation soil sieve analysis and classification, 10.82-11.82 ft

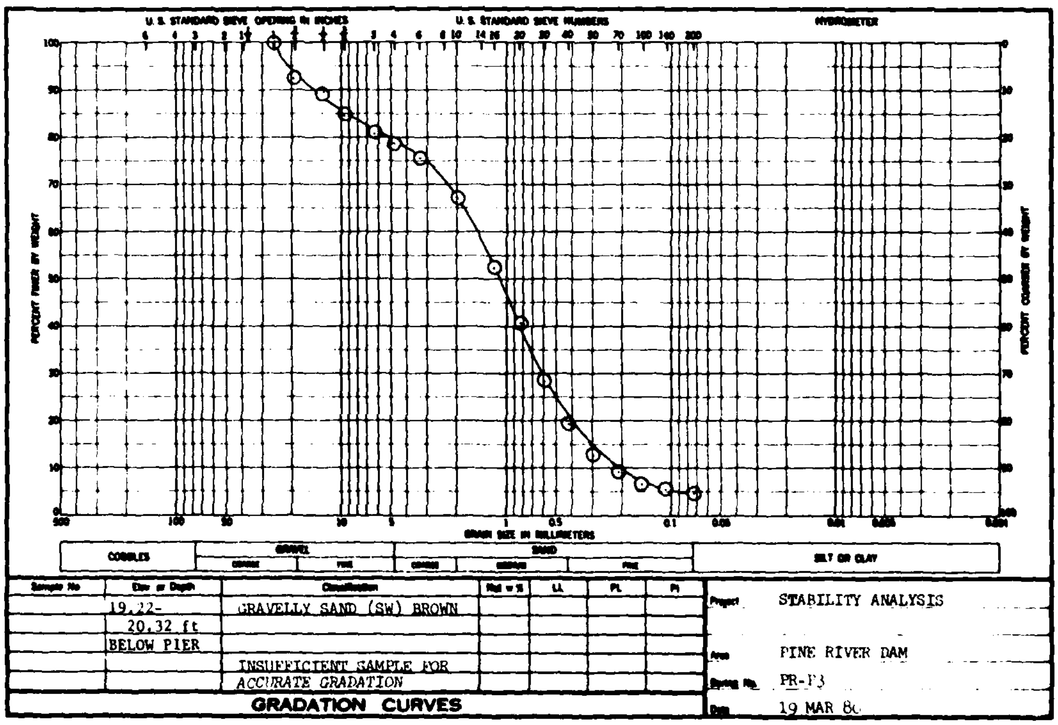
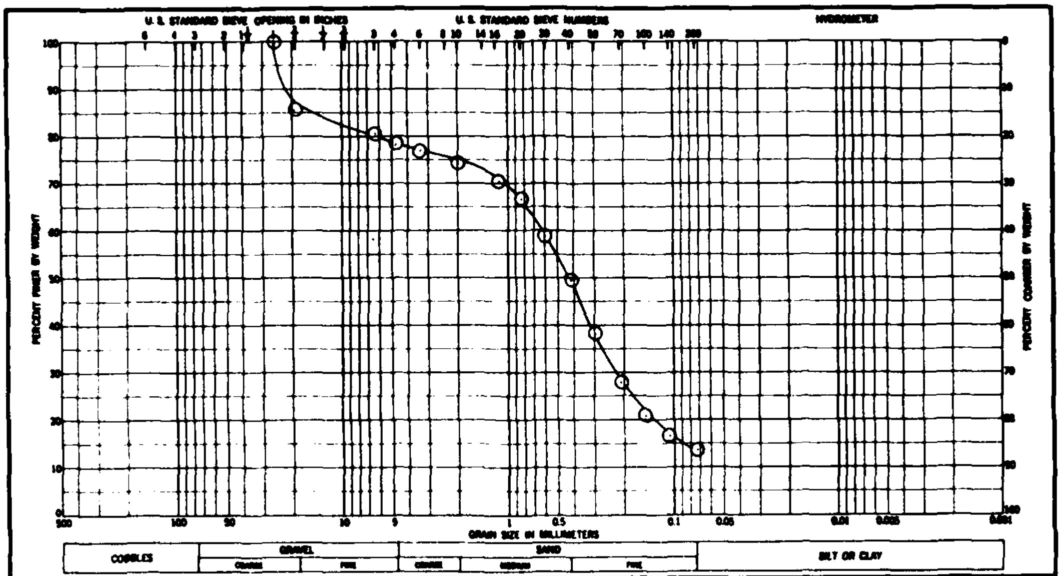


Figure 17. Foundation soil sieve analysis and classification, 19.22-20.32 ft

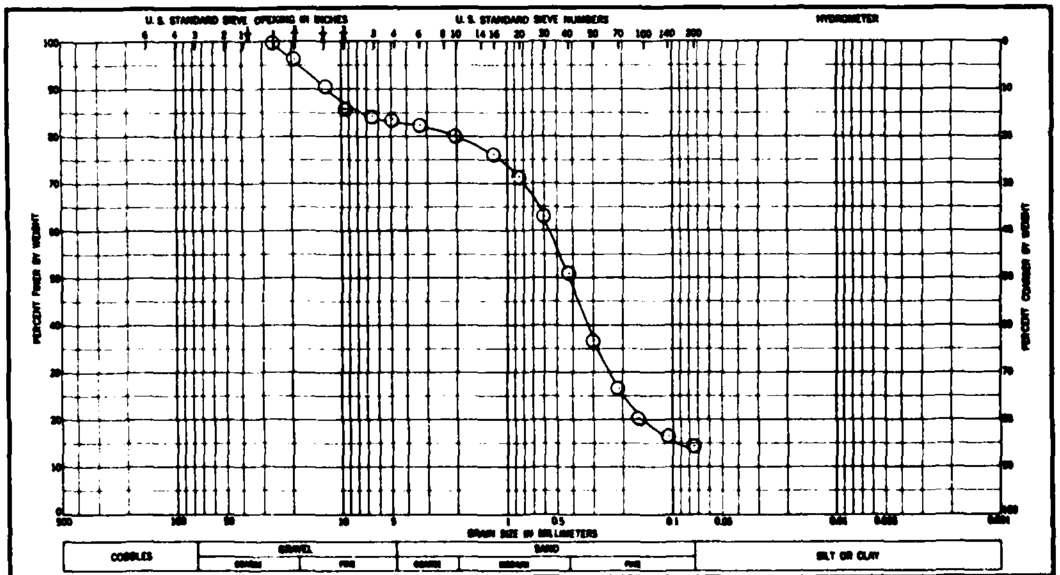


Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
	0.62-2.04 ft	GRAVELLY SILTY SAND (SM)				
	BELOW PIER	BROWN				
		INSUFFICIENT SAMPLE FOR				
		ACCURATE GRADATION				

Project: STABILITY ANALYSIS
Area: PINE RIVER DAM
Boring No.: PR-P6
Date: 19 MAR 80

GRADATION CURVES

Figure 18. Foundation soil sieve analysis and classification, 0.62-2.04 ft



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
	9.52-	GRAVELLY SILTY SAND (SM)				
	11.02 ft	BROWN				
	BELOW PIER					
		INSUFFICIENT SAMPLE FOR				
		ACCURATE GRADATION				

Project: STABILITY ANALYSIS
Area: PINE RIVER DAM
Boring No.: PR-P6
Date: 19 MAR 80

GRADATION CURVES

Figure 19. Foundation soil sieve analysis and classification, 9.52-11.02 ft

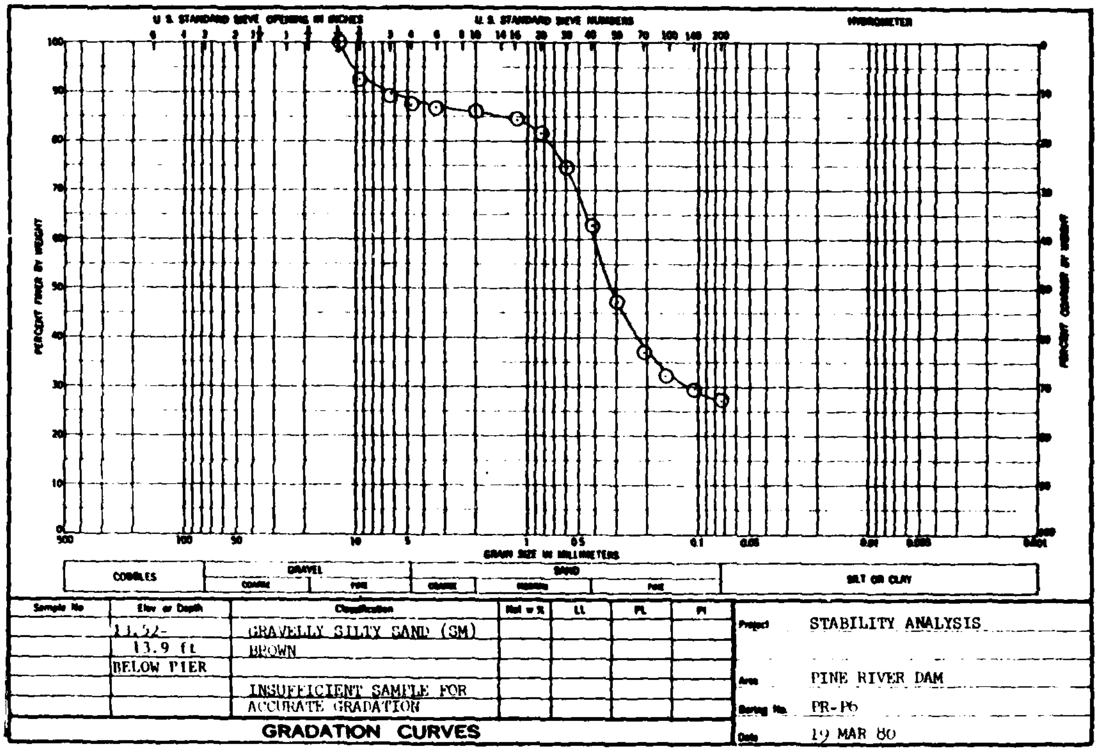


Figure 20. Foundation soil sieve analysis and classification, 13.52-13.9 ft

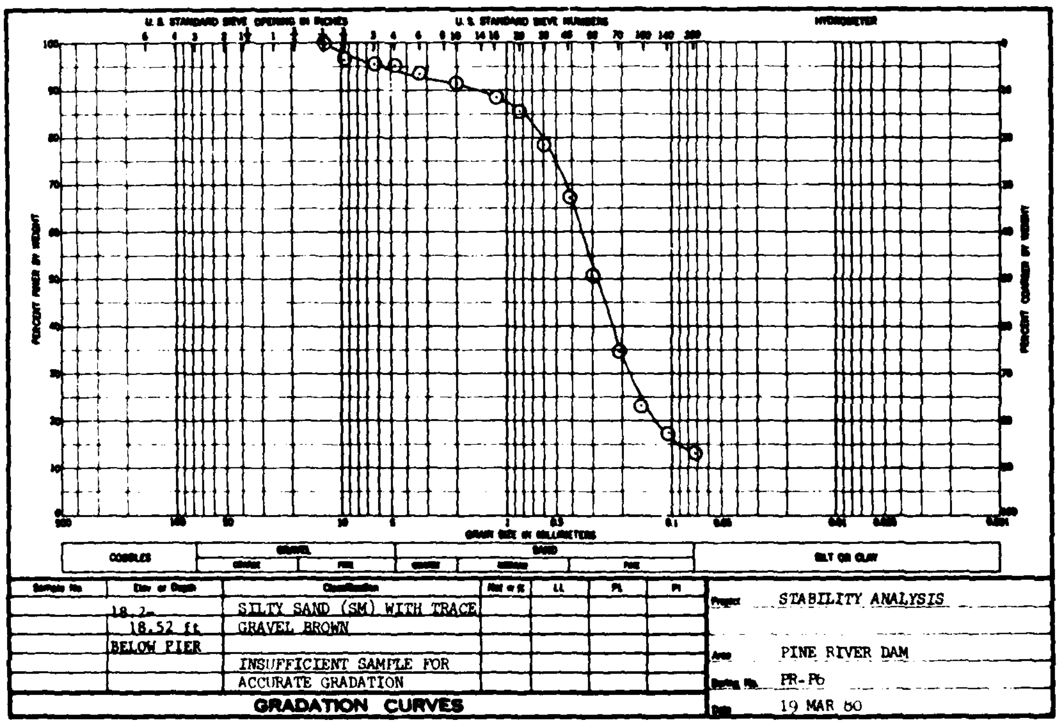


Figure 21. Foundation soil sieve analysis and classification, 18.2-18.52 ft

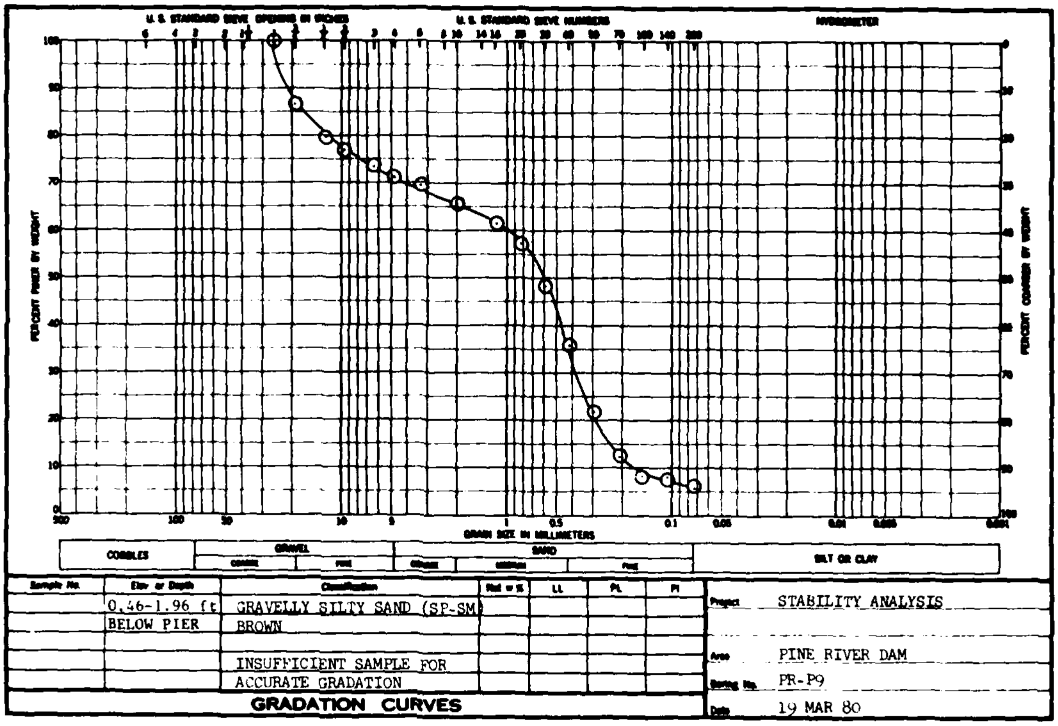


Figure 22. Foundation soil sieve analysis and classification, 0.46-1.96 ft

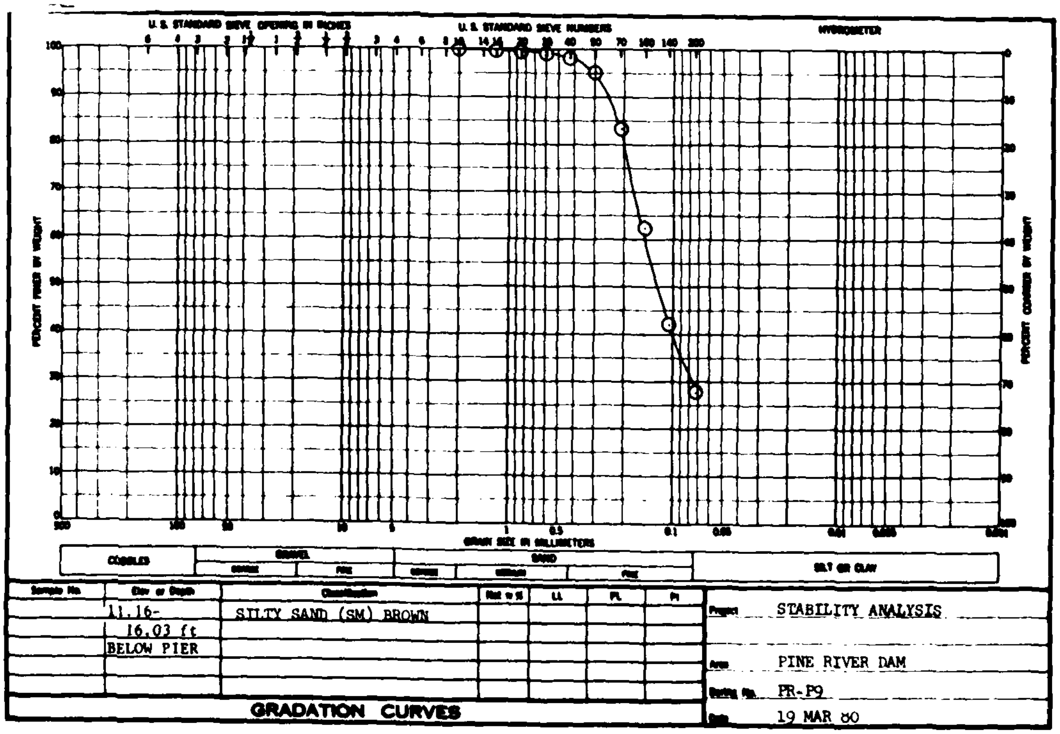


Figure 23. Foundation soil sieve analysis and classification, 11.16-16.03 ft

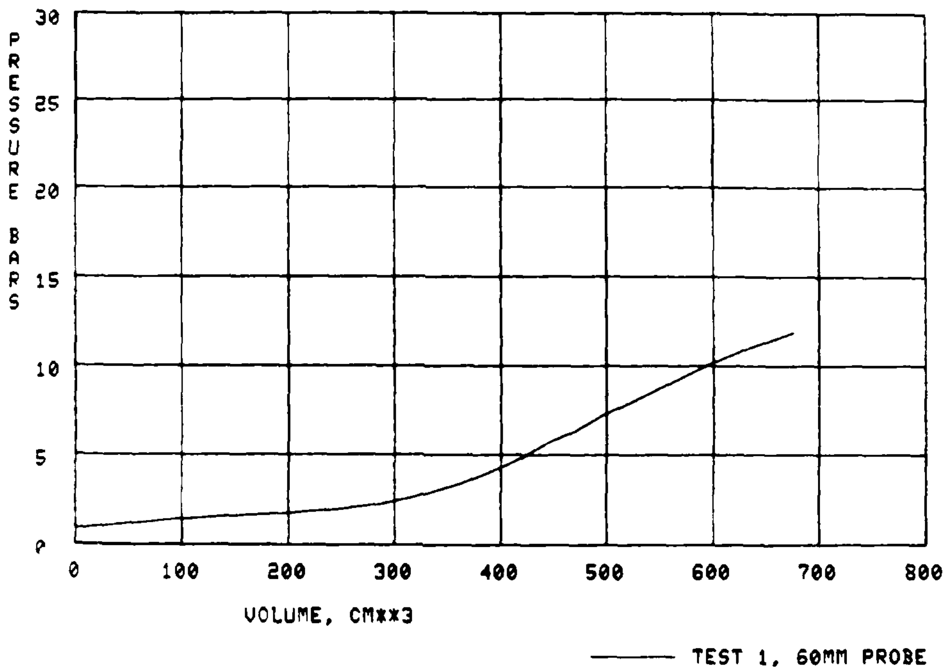


Figure 24. Pressure versus volume (metric units), PR-P3

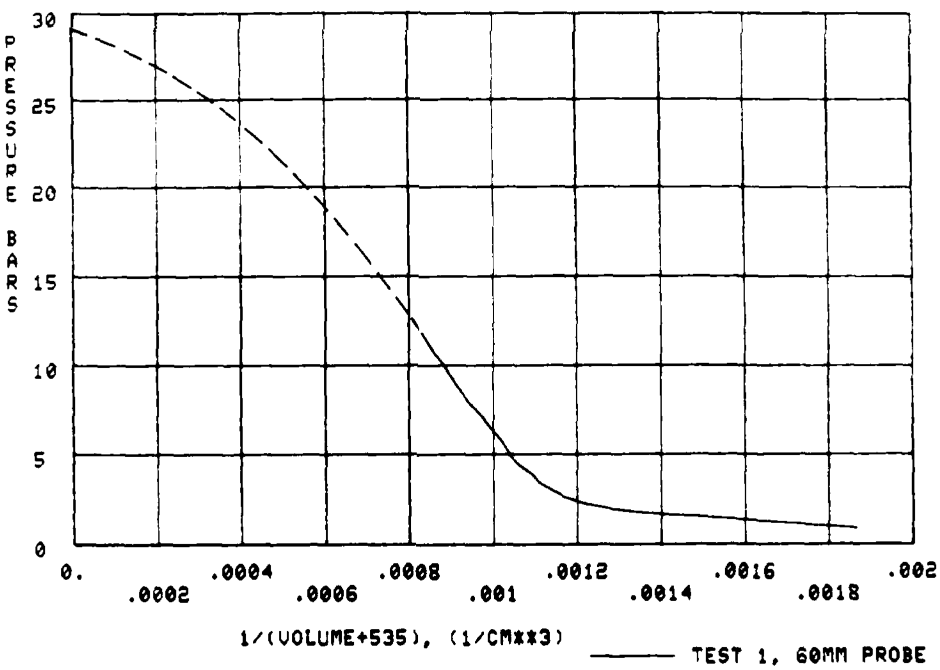


Figure 25. Limit pressure determination, PR-P3

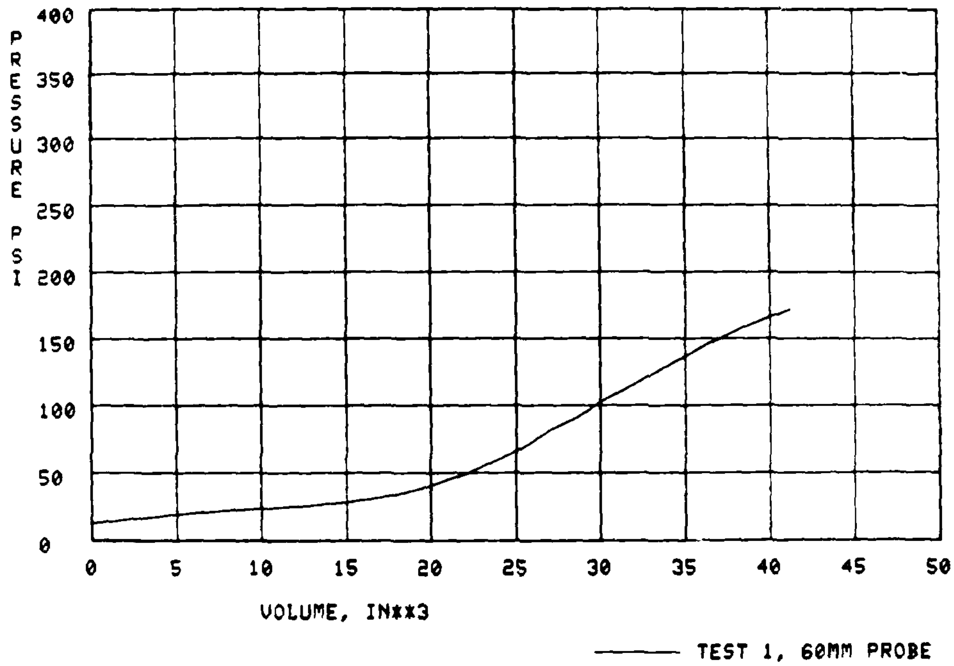


Figure 26. Pressure versus volume, PR-P3

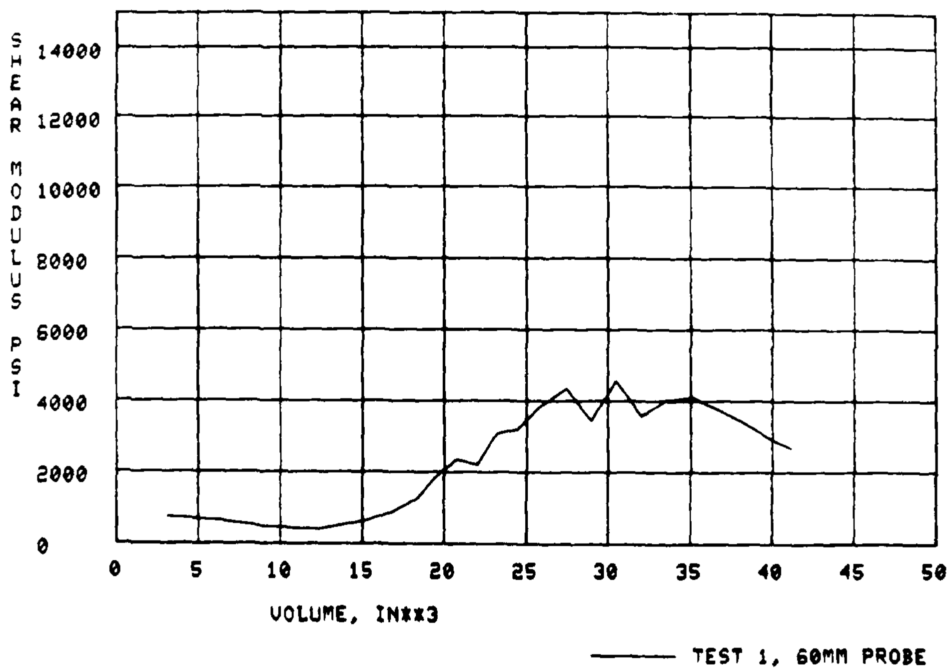


Figure 27. Shear modulus, PR-P3

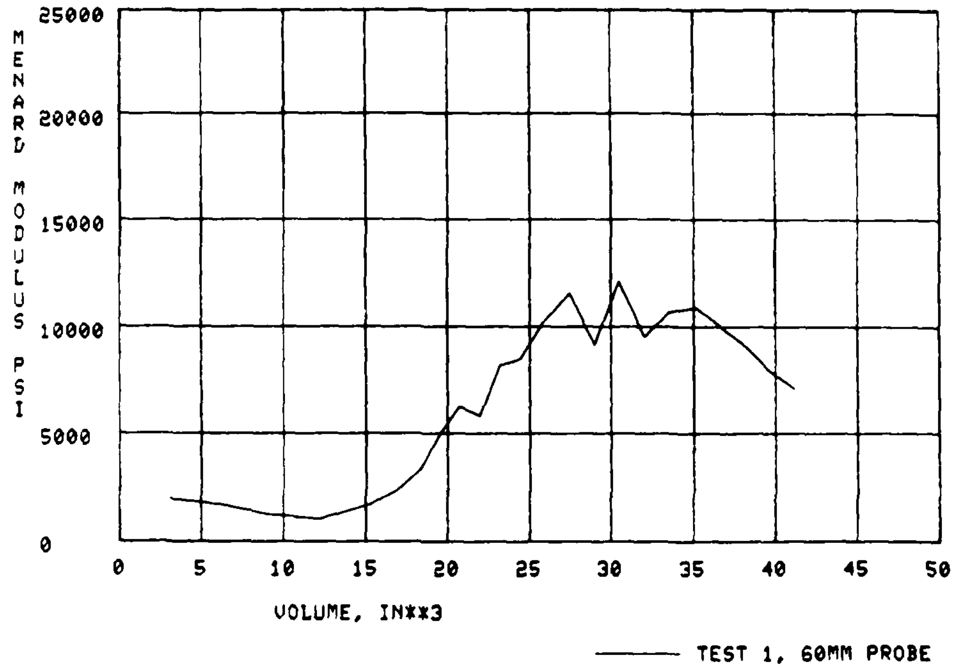


Figure 28. Ménard modulus, PR-P3

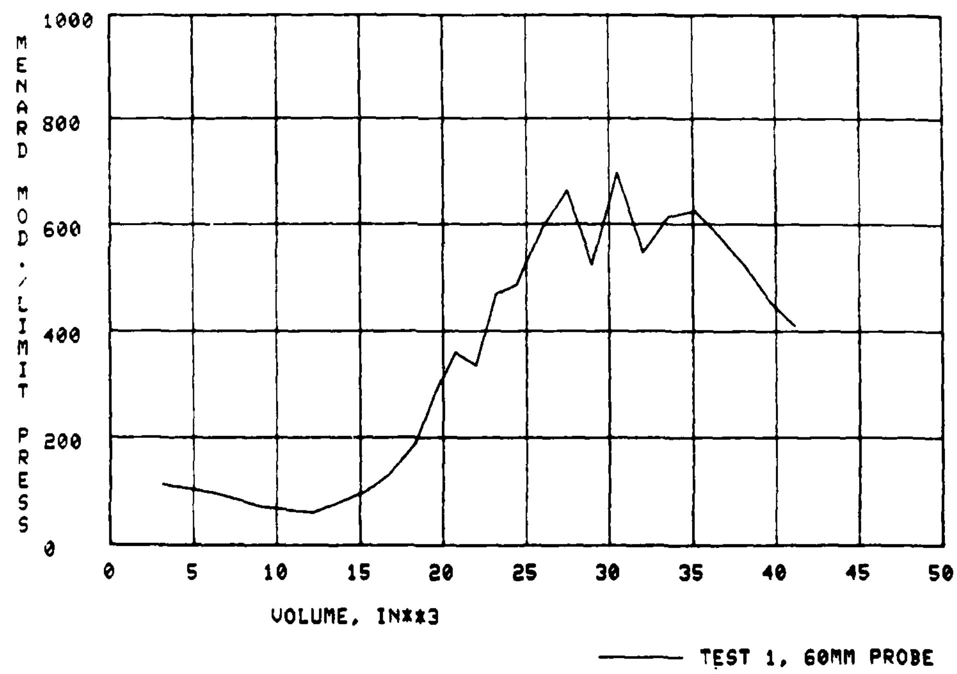


Figure 29. Ménard modulus divided by limit pressure, PR-P3

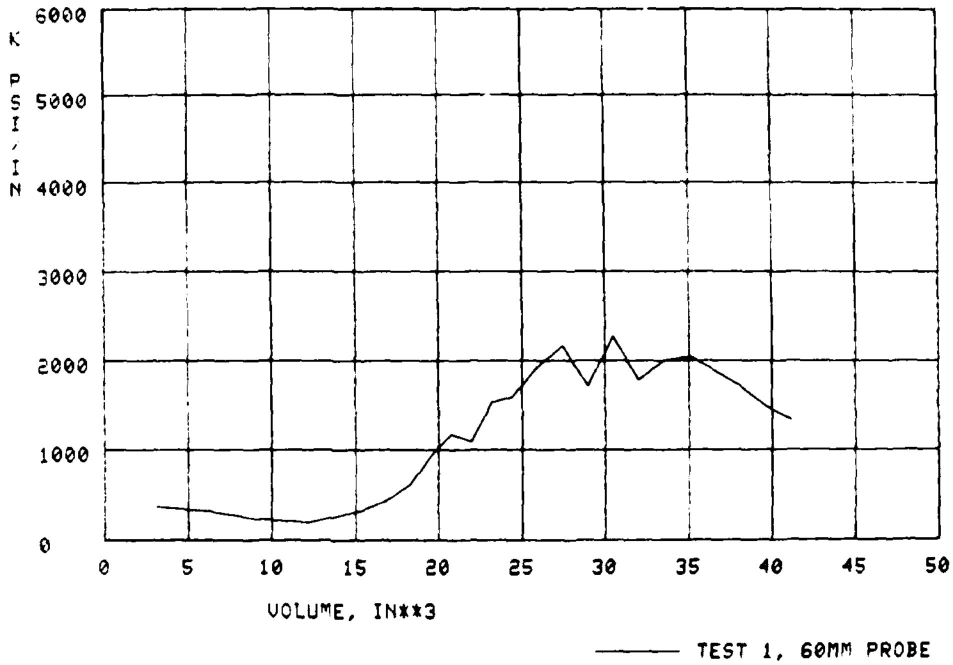


Figure 30. Horizontal subgrade modulus, PR-P3

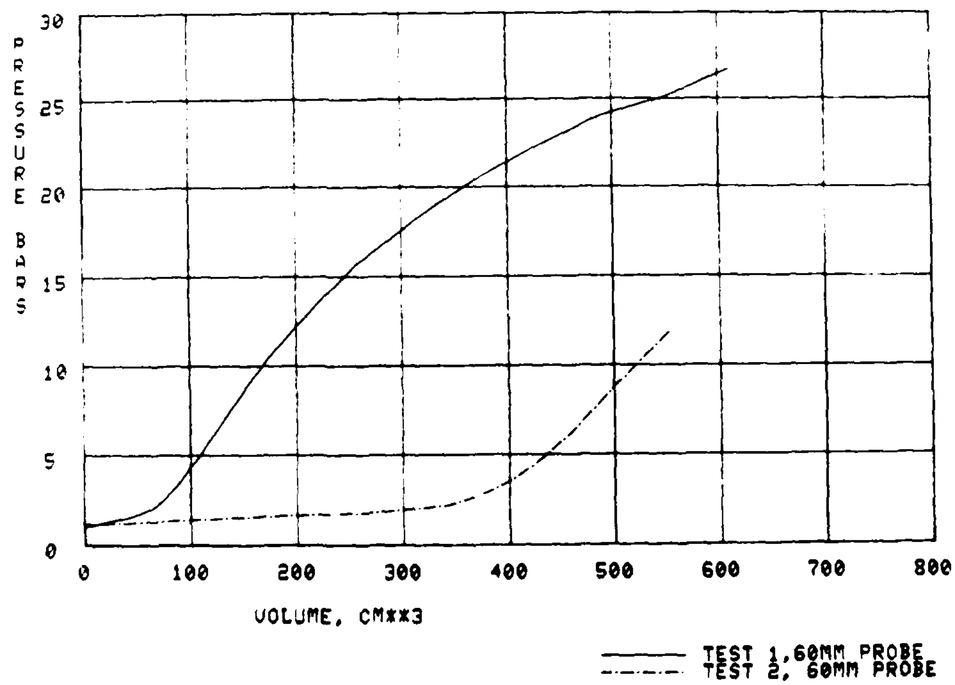


Figure 31. Pressure versus volume (metric units), PR-P6

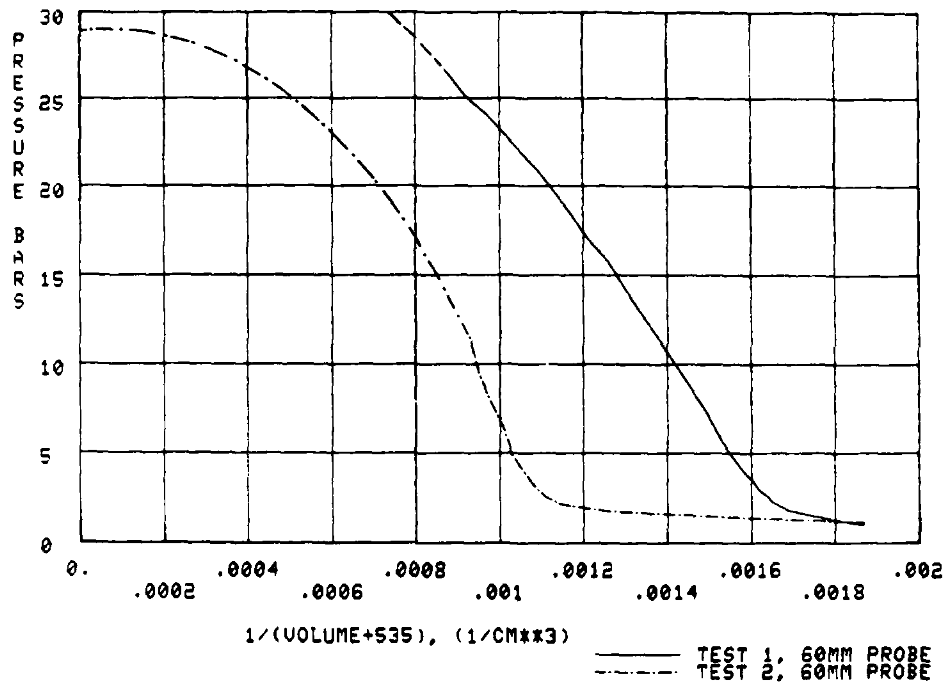


Figure 32. Limit pressure determination, PR-P6

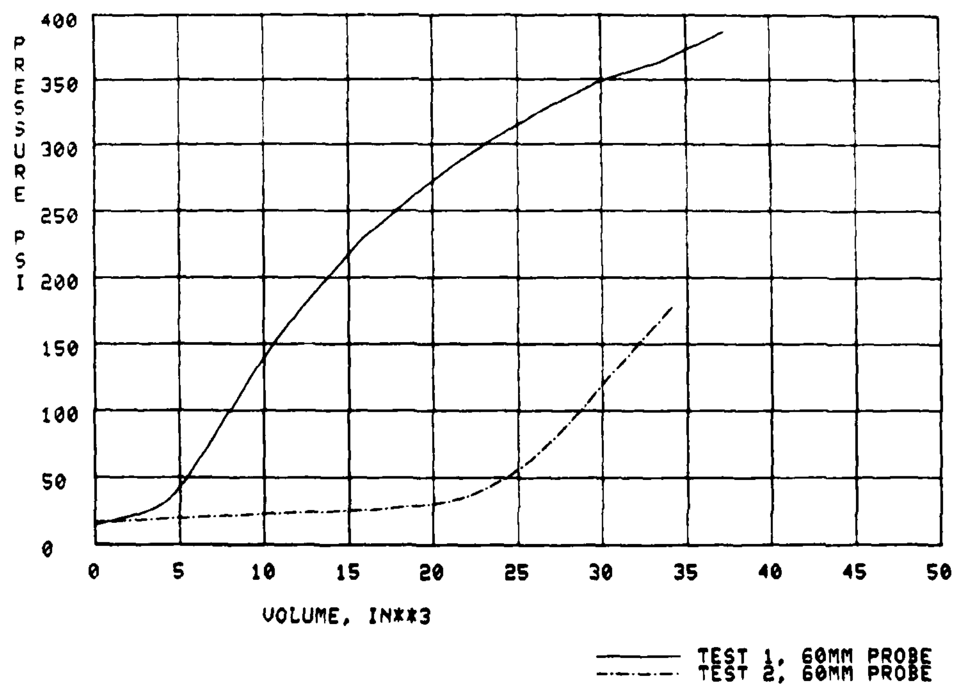


Figure 33. Pressure versus volume, PR-P6

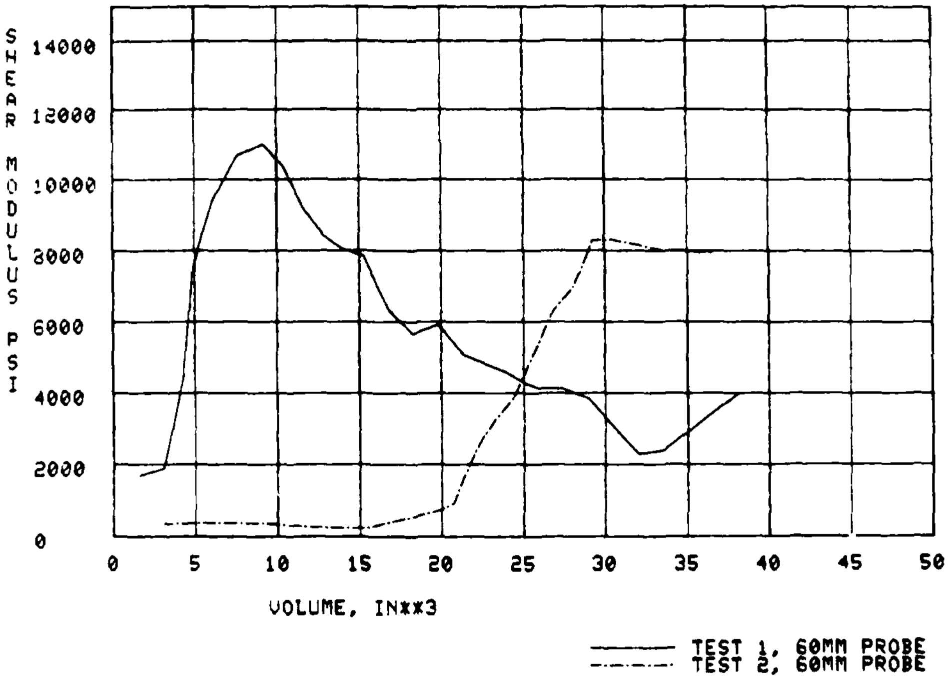


Figure 34. Shear modulus, PR-P6

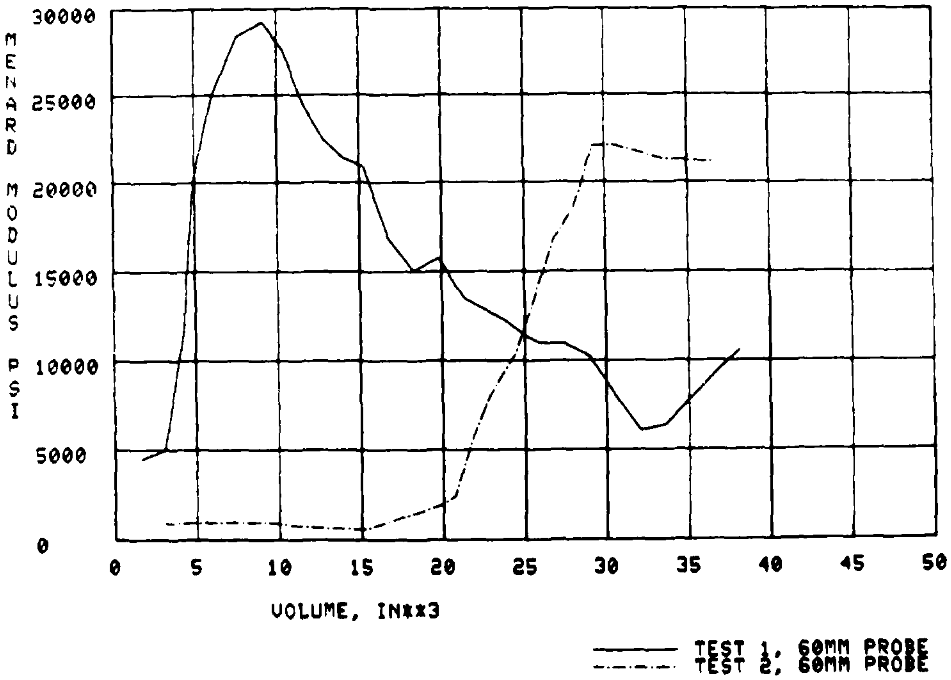


Figure 35. Ménard modulus, PR-P6

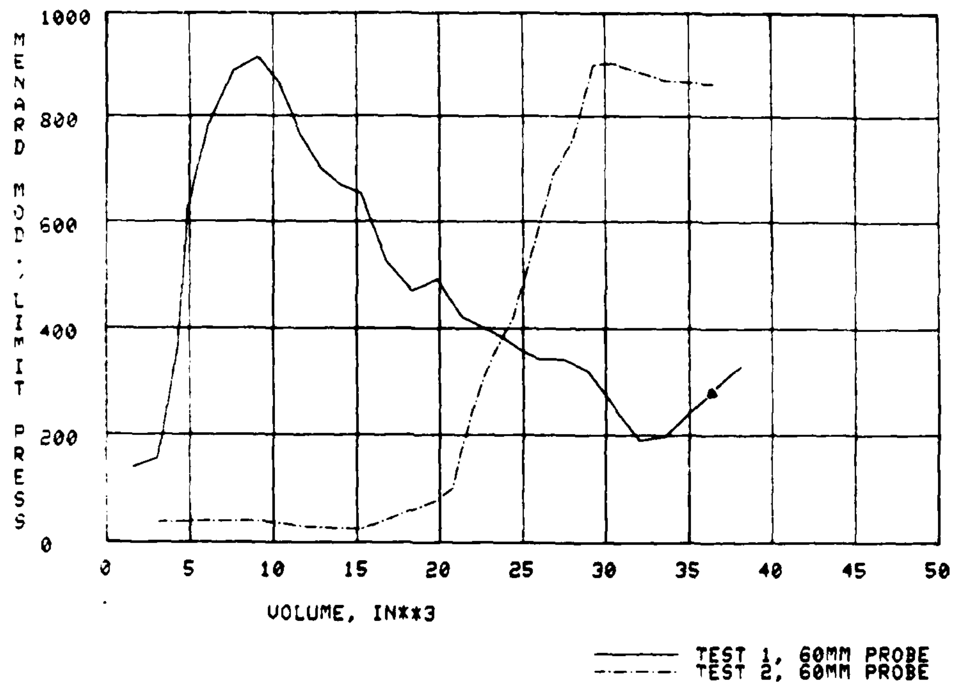


Figure 36. Ménard modulus divided by limit pressure, PR-P6

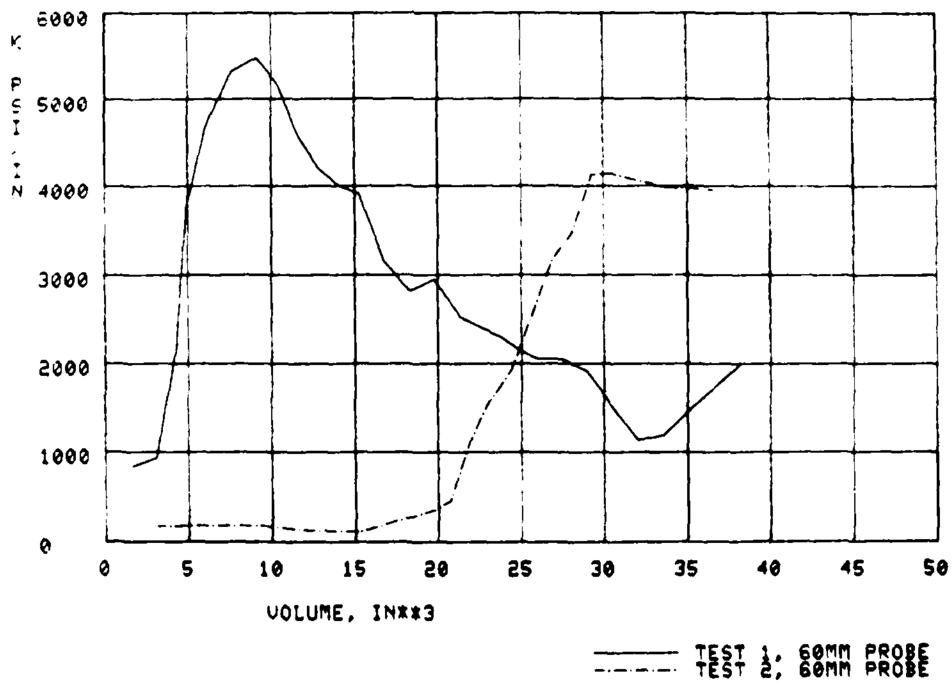


Figure 37. Horizontal subgrade modulus, PR-P6

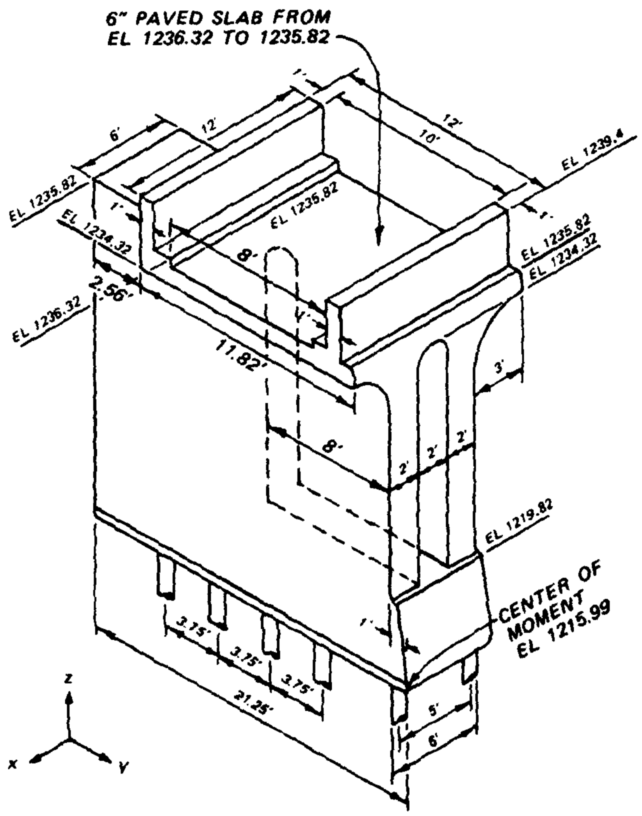


Figure 38. Schematic presenting geometry of interior monolith, Pine River Dam

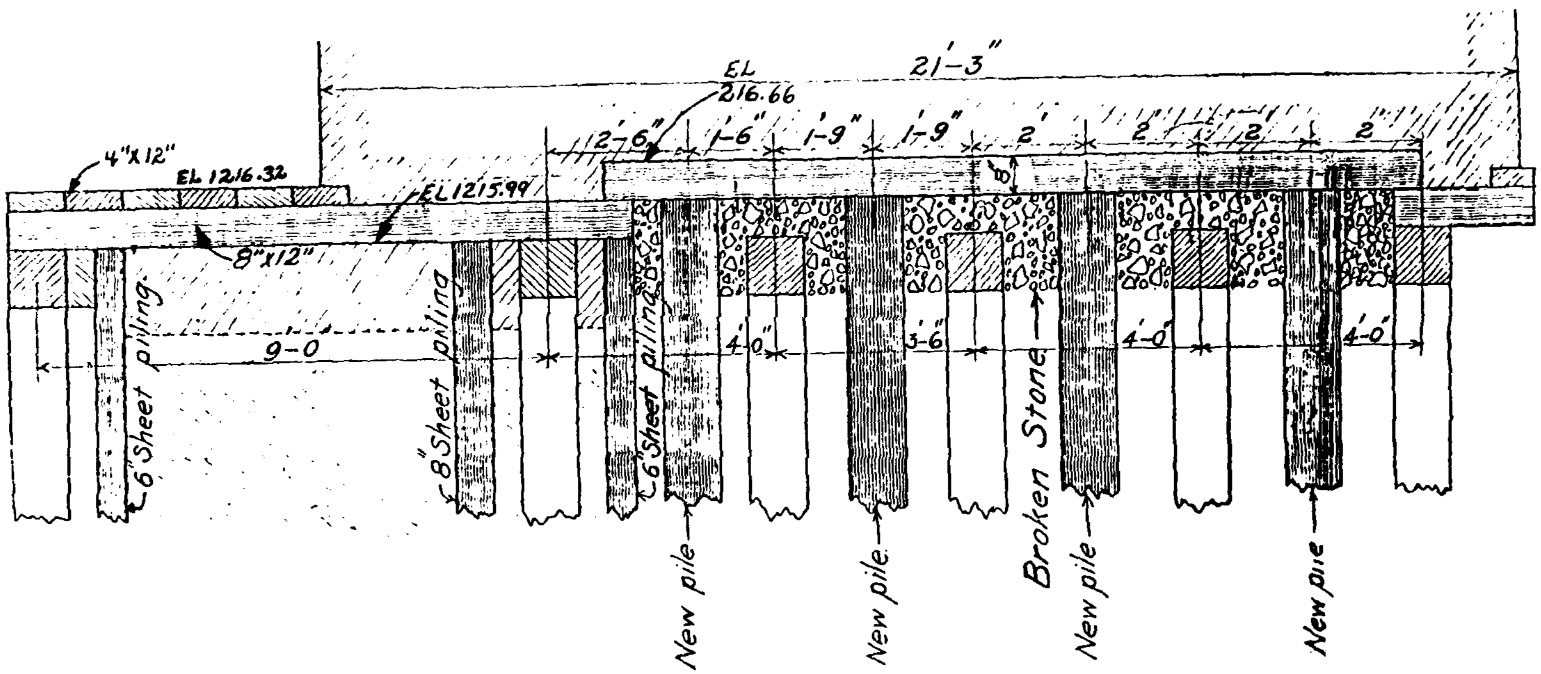


Figure 39. Section view through pier

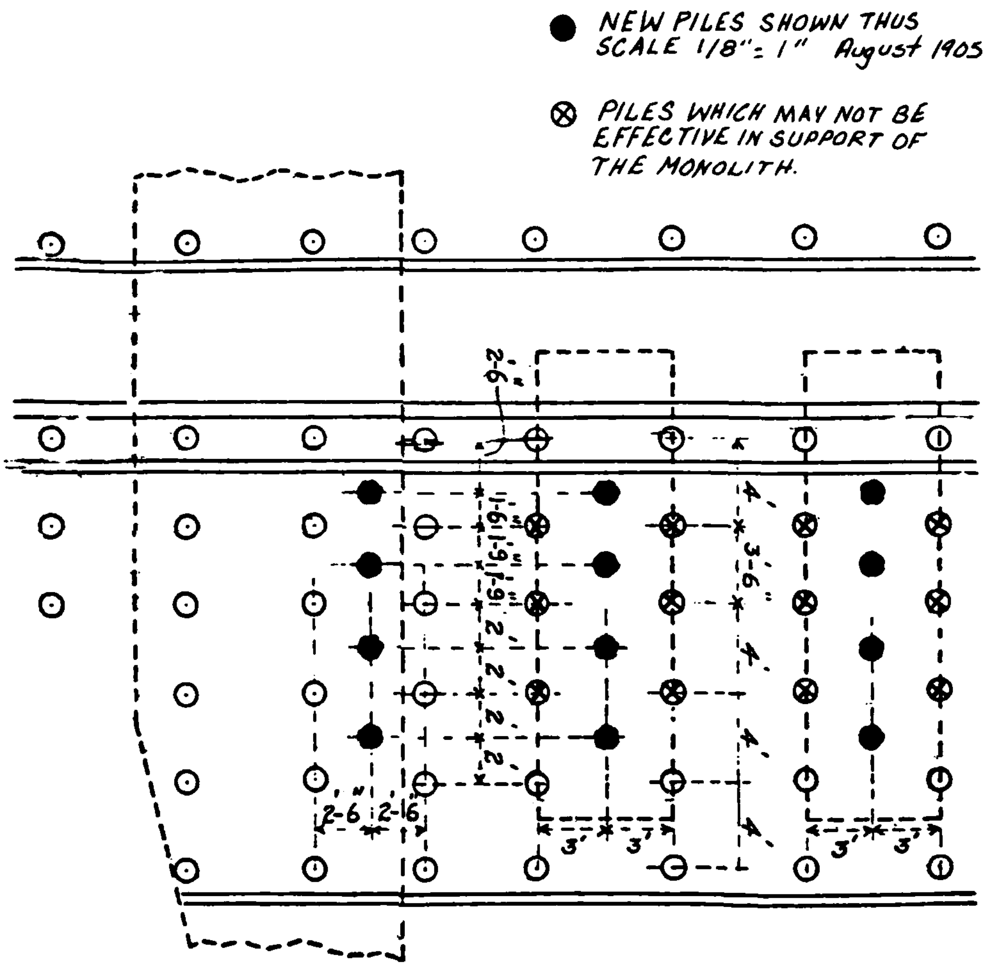


Figure 40. Plan view, south end of dam, showing location of piles

Item	Factor	F_y (kips)	F_z (kips)	Arm_y (ft)	Arm_z (ft)	M_x (ft-k)
W_{Conc}	$(0.15)(1239.4-1235.82)(2)(12)$		12.89	13.00		167.6
	$(0.15)(1236.32-1235.82)(10)(12)$		9.00	13.00		117.0
	$(0.15)(1235.82-1234.32)(12.44)(6)$		16.79	12.78		214.6
	$(0.15)(1234.32-1231.32)(13.37)(6)$		36.10	12.32		444.8
	$-(0.15)(1/2)(\pi)(3)^2(13.37)$		-28.35	12.32		-349.3
	$(0.15)(1235.82-1215.99)(14.38)(6)$		256.64	14.06		3608.4
	$(0.15)(1/2)(1235.82-1219.82)(6.62)(6)$		47.66	4.66		222.1
	$(0.15)(1219.82-1215.99)(6.87)(6)$		23.68	3.44		81.5
	$-(0.15)(1/2)(\pi)(1)^2(8)$		-1.88	10.04		-18.9
	$-(0.15)(1233.32-1219.82)(8)(2)$		-32.40	7.56		-244.9
			340.13			4242.9
$P_{Headwater}$	$-(0.0625)(1/2)(1231.12-1215.99)^2(12)$	-85.84			5.04	-432.6
Uplift	$-[16+3.75][\frac{(0.0625)(1231.12-1215.12)}{16+29+16}](21.25)(6)$		-41.28	10.63		-438.8
	$-(1/2)[29-4-3.75][\frac{(0.0625)(1231.12-1215.12)}{16+29+16}](21.25)(6)$		-22.21	14.17		-314.7
			-63.49			-753.5
	$e = \frac{3056.8}{276.64} = 11.05$ ft					
Total		-85.84	276.64			3056.8

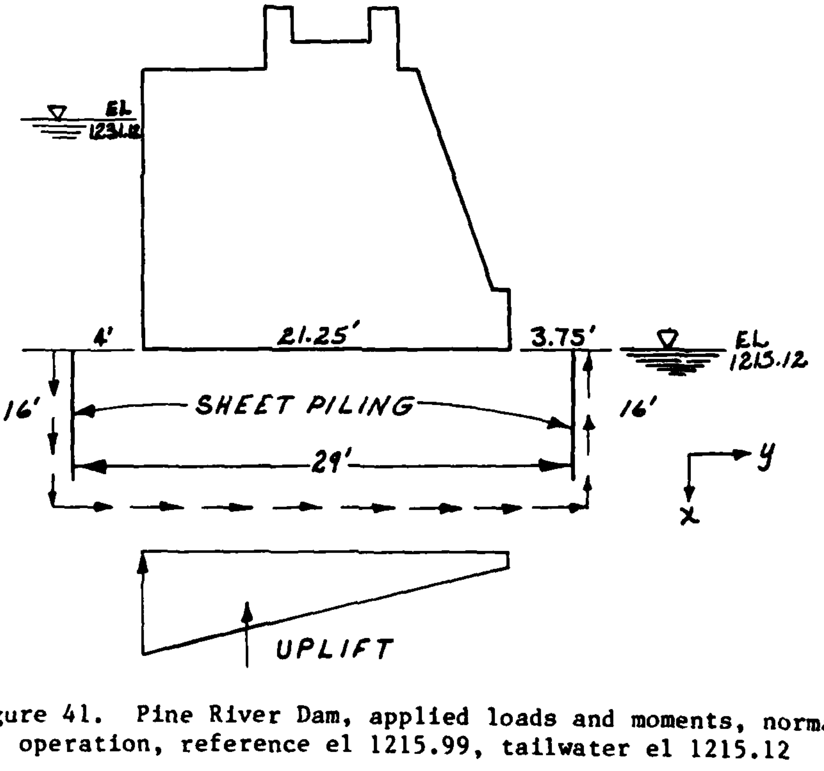


Figure 41. Pine River Dam, applied loads and moments, normal operation, reference el 1215.99, tailwater el 1215.12

Item	Factor	F_y (kips)	F_z (kips)	Arm_y (ft)	Arm_z (ft)	M_x (ft-k)
Loads	From normal operation calculations	-85.84	276.64			3056.8
P_{Truck}	H15-44 truck loading		24.00	13.00		312.0
Total		-85.84	300.64			3368.8

$$e = \frac{3368.8}{300.64} = 11.21 \text{ ft}$$

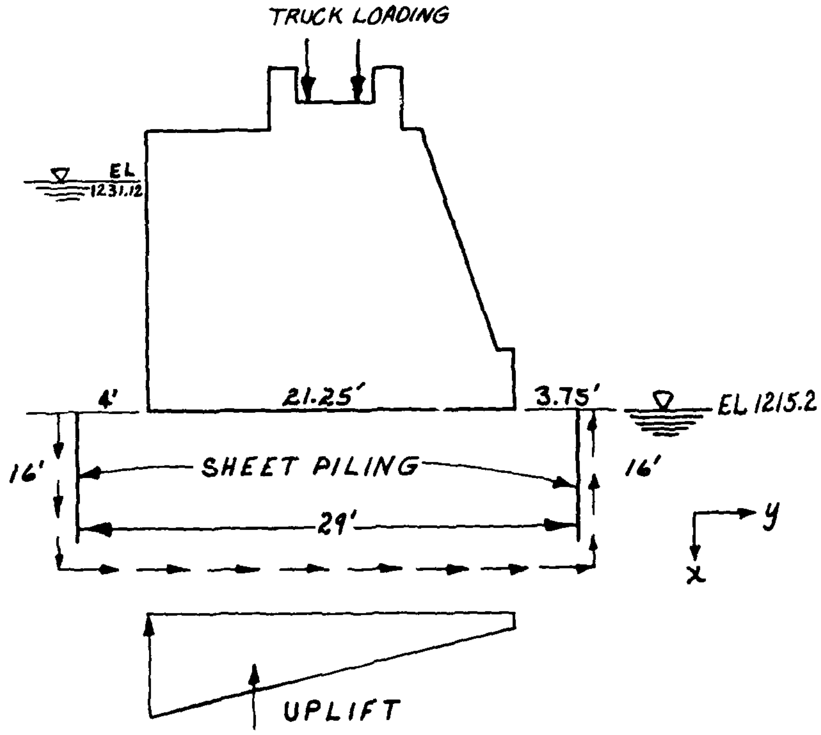


Figure 42. Pine River Dam, applied loads and moments, normal operation with truck loading (H15-44), reference el 1215.99, tailwater el 1215.2

Item	Factor	F_y (kips)	F_z (kips)	Arm_y (ft)	Arm_z (ft)	M_x (ft-k)
Loads	From normal operation calculations	-85.84	276.64			3056.8
Earthquake:						
Pe_1	(0.025)(340.13)	-8.50			10.49	-89.2
Pe_2	$(2/3)(51)(0.025)(15.13)^2(12)(1/1000)$	-2.33			6.05	-14.1
Total		-96.67	276.64			2953.5

$$e = \frac{2953.5}{276.64} = 10.68 \text{ ft}$$

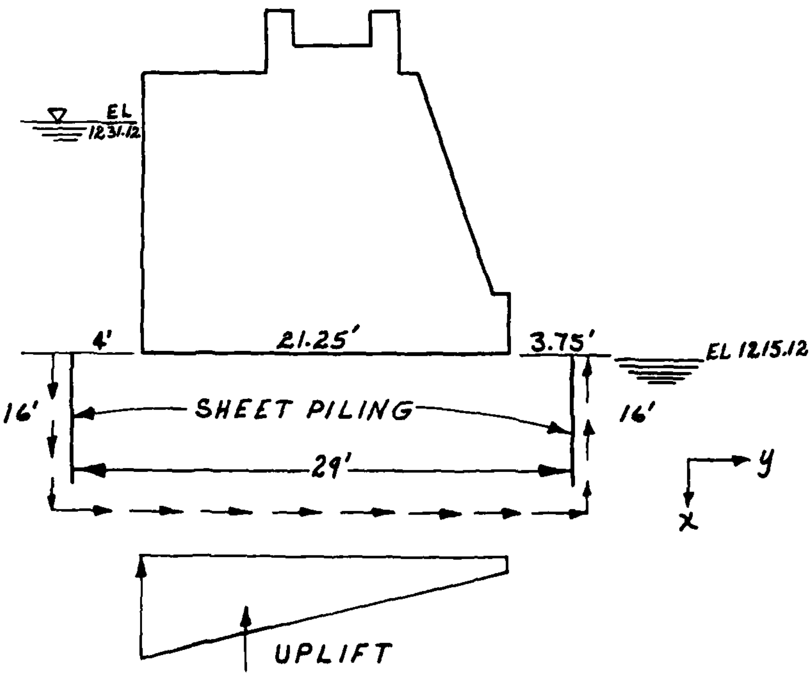


Figure 43. Pine River Dam, applied loads and moments, normal operation with earthquake, reference el 1215.99, tailwater el 1215.12

LOCATION OF CENTER OF GRAVITY OF PIER IN XZ PLANE

(Bottom of pier at elevation 1215.99)

$(12.89)[(1/2)(1239.4 - 1235.82) + 1235.82 - 1215.99]$	=	278.7
$(9)[(1/2)(1236.32 - 1235.82) + 1235.82 - 1215.99]$	=	180.7
$(16.79)[(1/2)(1235.82 - 1234.32) + 1234.32 - 1215.99]$	=	320.4
$(36.10)[(1/2)(1234.32 - 1231.32) + 1231.32 - 1215.99]$	=	607.6
$(-28.35)[(\frac{4}{3\pi})(1234.32 - 1231.32) + 1231.32 - 1215.99]$	=	-470.7
$(256.65)(1/2)(1235.82 - 1215.99)$	=	2544.6
$(47.66)[(1/3)(1235.82 - 1219.82) + 1219.82 - 1215.99)$	=	436.7
$(23.68)(1/2)(1219.82 - 1215.99)$	=	45.4
$-(1.88)[\frac{4}{3\pi} + 1233.32 - 1215.99]$	=	-33.4
$-(32.4)[(1/2)(1233.32 - 1219.82) + 1219.82 - 1215.99$	=	-342.8
		3567.2

$$\bar{Y} = \frac{3567.2}{340.13} = 10.49 \text{ ft}$$

Figure 44. Pine River Lake Dam, normal operation with earthquake, location of center of gravity of pier in XZ plane

Item	Factor	F_y (kips)	F_z (kips)	Arm_y (ft)	Arm_z (ft)	M_x (ft-k)
Loads	From normal operation calculations	-85.84	276.64			3056.8
P_{Ice}	(1)(5)(12)	-60.00			14.63	-877.8
Total		-145.84	276.64			2179.0

$$e = \frac{2179.0}{276.64} = 7.88 \text{ ft}$$

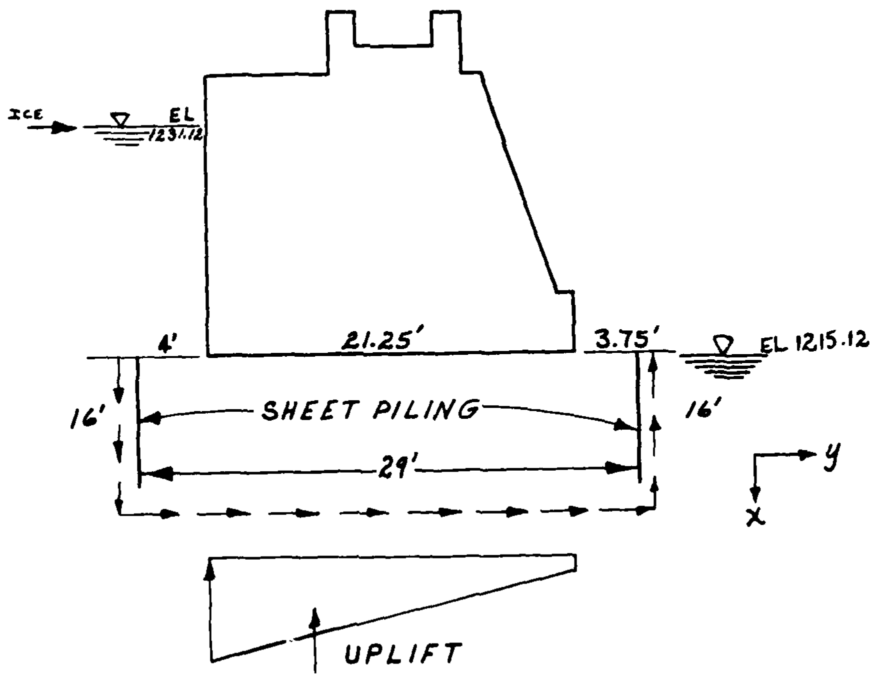


Figure 45. Pine River Dam, normal operation with ice, reference el 1215.99, tailwater el 1215.12

Item	Factor	F_y (kips)	F_z (kips)	Arm_y (ft)	Arm_z (ft)	M_x (ft-k)
W_{Conc}	From normal operation calculations		340.13			4242.9
$P_{Headwater}$	$-(0.0625)(1/2)(1235.12-1215.99)^2(12)$	-137.23			6.38	-875.5
$P_{Tailwater}$	$-(0.0625)(1/2)(1219.12-1215.99)^2(12)(0.6)$	2.20			1.04	2.3
Uplift	$-(0.0625)(1219.12-1215.99)(21.25)(6)$		-24.94	10.63		-265.1
	$-[16+3.75][\frac{(0.0625)(1235.12-1219.12)}{16+29+16}](21.25)(6)$		-41.28	10.63		-438.8
	$-(1/2)[29-4-3.75][\frac{(0.0625)(1235.12-1219.12)}{16+29+16}](21.25)(6)$		-22.21	14.17		-314.7
			-88.43			-1018.6

$$e = \frac{2351.1}{251.7} = 9.34 \text{ ft}$$

Total		-135.03	251.70			2351.1

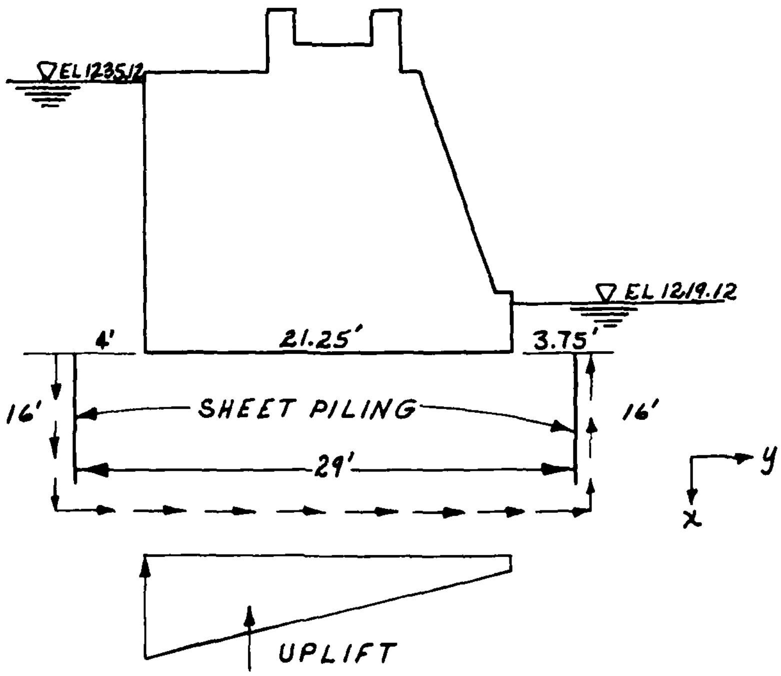


Figure 46. Pine River Dam, high-water condition, reference el 1215.99, tailwater el 1219.12

Item	Factor	F_y (kips)	F_z (kips)	Arm_y (ft)	Arm_z (ft)	M_x (ft-k)
W_{Conc}	$(0.15)(1239.4-1235.82)(2)(12)$		12.89	13.00		167.6
	$(0.15)(1236.32-1235.82)(10)(12)$		9.00	13.00		117.0
	$(0.15)(1235.82-1234.32)(12.44)(6)$		16.79	12.78		214.6
	$(0.15)(1234.32-1231.32)(13.37)(6)$		36.10	12.32		444.8
	$-(0.15)(1/2)(\pi)(3)^2(13.37)$		-28.35	12.32		-349.3
	$(0.15)(1235.82-1214.32)(14.38)(6)$		278.25	14.06		3912.2
	$-(0.15)(1/2)(1235.82-1219.82)(6.62)(6)$		47.66	4.66		222.1
	$(0.15)(1219.82-1214.32)(6.87)(6)$		34.01	3.44		117.0
	$-(0.15)(1/2)(\pi)(1)^2(8)$		-1.88	10.04		-18.9
	$-(0.15)(1233.32-1219.82)(8)(2)$		-32.40	7.56		-244.9
			372.06			4582.2
$P_{Headwater}$	$-(0.0625)(1/2)(1231.12-1214.32)^2(12)$	-105.84			5.60	-592.7
$P_{Tailwater}$	$(0.0625)(1/2)(1215.12-1214.32)^2(12)$	0.24			0.27	0.1
Uplift	$-[16+3.75][\frac{(0.0625)(1231.12-1215.12)}{16+29+16}](21.25)(6)$		-41.28	10.63		-438.8
	$-(1/2)[29-4-3.75][\frac{(0.0625)(1231.12-1215.12)}{16+29+16}](21.25)(6)$		-22.21	14.17		-314.7
			-63.49			-753.5
	$e = \frac{3236.1}{308.57} = 10.49$ ft					
Total		-105.60	308.57			3236.1

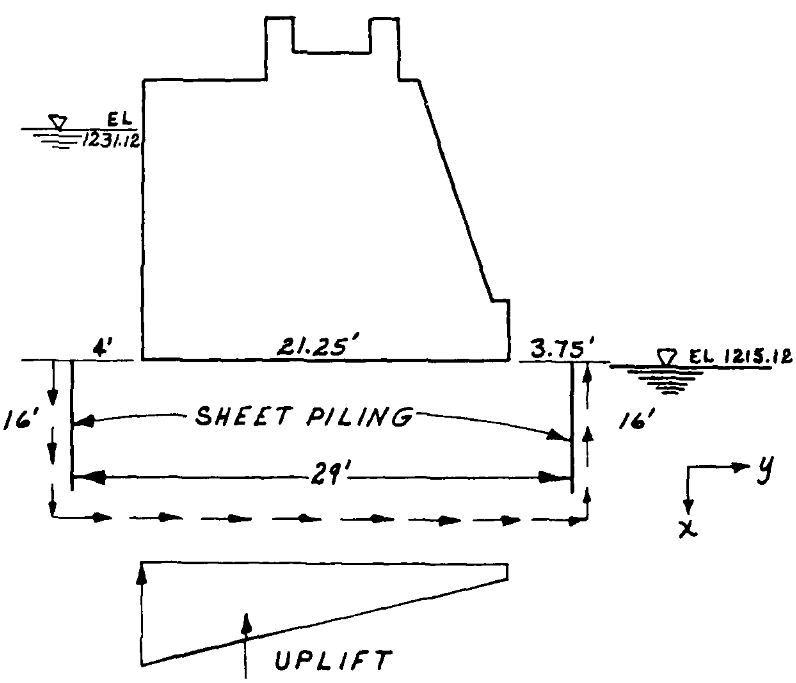


Figure 47. Pine River Dam, normal operation, reference el 1214.32, tailwater el 1215.12

Item	Factor	F_y (kips)	F_z (kips)	Arm_y (ft)	Arm_z (ft)	M_x (ft-k)
Loads	From normal operation calculations	-105.60	308.57			3236.1
P_{Truck}	H15-44 truck loading		24.00	13.00		312.0
	$e = \frac{3548.1}{332.57} = 10.67$ ft					
Total		-105.60	332.57			3548.1

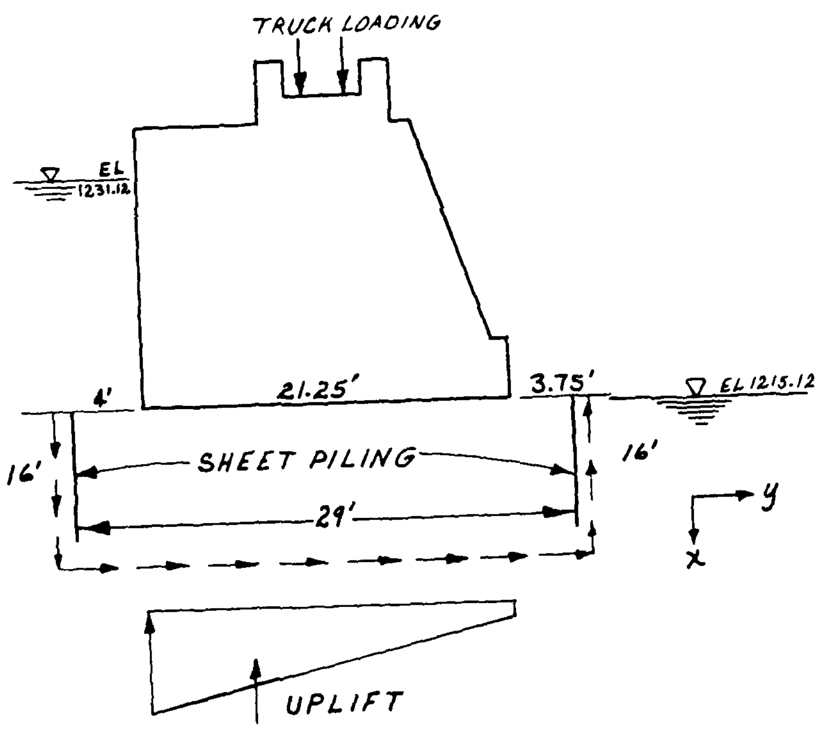


Figure 48. Pine River Dam, normal operation with truck loading (H15-44), reference el 1214.32, tailwater el 1215.12

Item	Factor	F_y (kips)	F_z (kips)	Arm_y (ft)	Arm_z (ft)	M_x (ft-k)
Loads	From normal operation calculations	-105.60	308.57			3236.1
Earthquake:						
Pe_1	(0.025)(372.06)	-9.30			11.19	-104.1
Pe_2	$(2/3)(51)(0.025)(16.80)^2(12)(1/1000)$	-2.88			6.72	-19.4
	$e = \frac{3112.6}{308.57} = 10.09$ ft					
Total		-117.78	308.57			3112.6

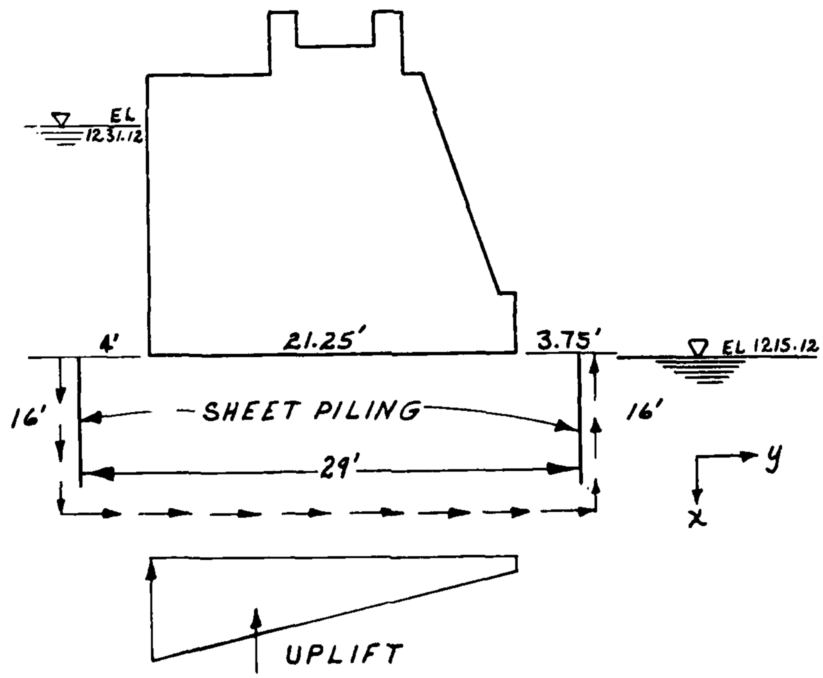


Figure 49. Pine River Dam, normal operation with earthquake, reference el 1214.32, tailwater el 1215.12

LOCATION OF CENTER OF GRAVITY OF PIER IN XZ PLANE
(Bottom of pier at elevation 1214.32)

$(12.89)[(1/2)(1239.4 - 1235.82) + 1235.82 - 1214.32]$	=	300.2
$(9)[(1/2)(1236.32 - 1235.82) + 1235.82 - 1214.32]$	=	195.8
$(16.79)[(1/2)(1235.82 - 1234.32) + 1234.32 - 1214.32]$	=	348.4
$(36.1)[(1/2)(1234.32 - 1231.32) + 1231.32 - 1214.32]$	=	667.8
$(-28.35)[\frac{4}{3\pi}(1234.32 - 1231.22) + 1231.32 - 1214.32]$	=	-518.0
$(278.27)[1/2][1235.82 - 1214.32]$	=	2991.4
$(47.66)[(1/3)(1235.82 - 1219.82) + 1219.82 - 1214.32]$	=	516.3
$(34.01)[1/2][1219.82 - 1214.32]$	=	93.5
$(-1.88)[\frac{4}{3\pi} + 1233.32 - 1214.32]$	=	-36.5
$(-32.4)[(1/2)(1233.32 - 1219.82) + 1219.82 - 1214.32]$	=	-396.9
		4162.0

$$Y = \frac{4162.0}{372.06} = 11.19 \text{ ft}$$

Figure 50. Pine River Lake Dam, normal operation with earthquake, location of center of gravity of pier in XZ plane

Item	Factor	F_y (kips)	F_z (kips)	Arm_y (ft)	Arm_z (ft)	M_x (ft-k)
Loads	From normal operation calculations	-105.60	308.57			3236.1
P_{Ice}	(1)(5)(12)	-60.00			16.30	-978.0
Total		-165.60	308.57			2258.1

$$e = \frac{2258.1}{308.57} = 7.32 \text{ ft}$$

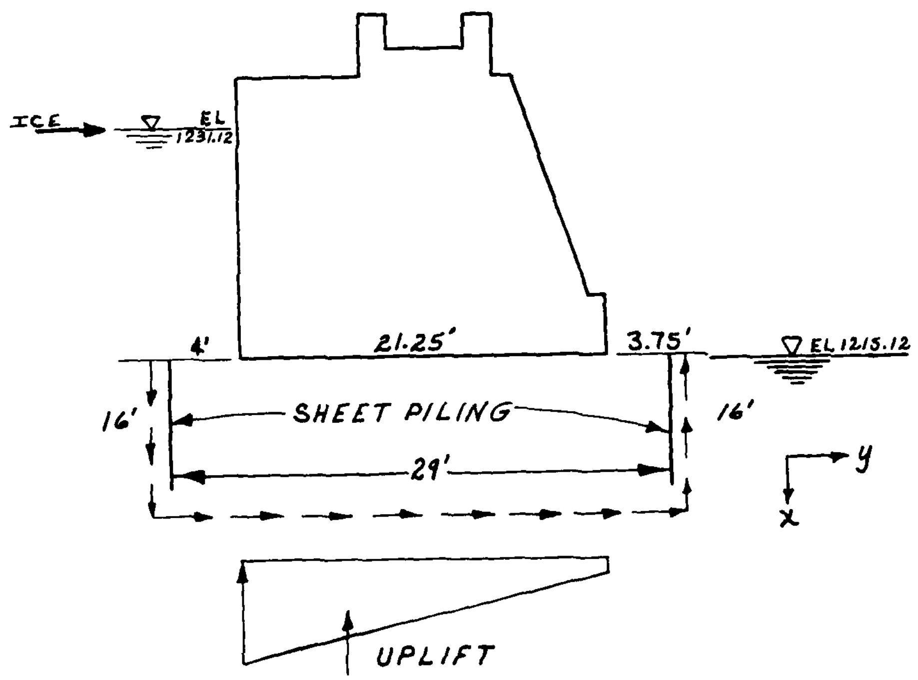


Figure 51. Pine River Dam, normal operation with ice, reference el 1214.32, tailwater el 1215.12

Item	Factor	F_y (kips)	F_z (kips)	Arm_y (ft)	Arm_z (ft)	M_x (ft-k)
W_{Conc}	From normal operation calculations		372.06			4582.2
$P_{Headwater}$	$-(0.0625)(1/2)(1235.12-1214.32)^2(12)$	-162.24			6.93	-1124.3
$P_{Tailwater}$	$(0.0625)(1/2)(1219.12-1214.32)^2(12)(0.6)$	5.18			1.60	8.3
	$-(0.0625)(1219.12-1215.32)(21.25)(6)$		-30.28	10.63		-321.9
	$-[16+3.75][\frac{(0.0625)(1235.12-1219.12)}{16+29+16}](21.25)(6)$		-41.28	10.63		-438.8
	$-(1/2)[29-4-3.75][\frac{(0.0625)(1235.12-1219.2)}{16+29+26}](21.25)(6)$		-22.21	14.17		-314.7
			-93.77			-1075.4

$$e = \frac{2390.8}{278.29} = 8.59 \text{ ft}$$

Total		-157.06	278.29			2390.8

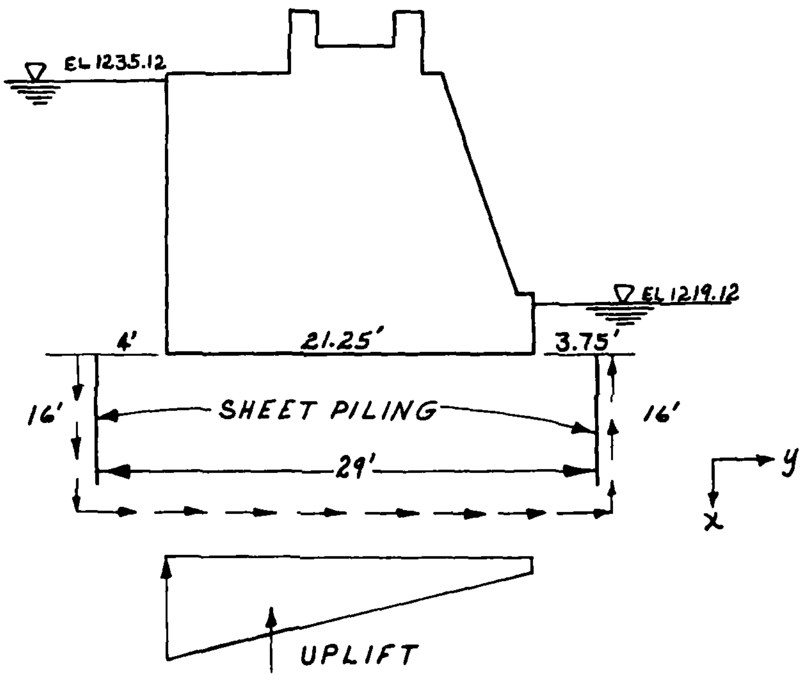


Figure 52. Pine River Dam, high-water condition, reference el 1214.32, tailwater el 1219.12

Item	Factor	F_y (kips)	F_z (kips)	Arm_y (ft)	Arm_z (ft)	M_x (ft-k)
W_{Conc}	$(0.15)(1239.4-1235.82)(2)(12)$		12.89	13.00		167.6
	$-(0.15)(1236.32-1235.82)(10)(12)$		9.00	13.00		117.0
	$(0.15)(1235.82-1234.32)(12.44)(6)$		16.79	12.78		214.6
	$(0.15)(1234.32-1231.32)(13.37)(6)$		36.10	12.32		444.8
	$-(0.15)(1/2)(\pi)(3)^2(13.37)$		-28.35	12.32		-349.3
	$(0.15)(1235.82-1215.99)(14.38)(6)$		256.64	14.06		3602.4
	$(0.15)(1/2)(1235.82-1219.82)(6.62)(6)$		147.66	4.66		222.1
	$(0.15)(1219.82-1215.99)(6.87)(6)$		23.68	3.44		81.5
	$-(0.15)(1/2)(\pi)(1)^2(8)$		-1.88	10.04		-18.9
	$-(0.15)(1233.32-1219.82)(8)(2)$		-32.40	7.56		-244.9
			340.13			4242.9
$P_{Headwater}$	$-(0.0625)(1/2)(1231.12-1215.99)^2(12)$	-85.84			5.04	-432.6
$P_{Tailwater}$	$(0.0625)(1/2)(1220.12-1215.99)^2(12)(0.6)$	3.84			1.38	5.3
Uplift	$(0.0625)(1220.12-1215.99)(21.25)(6)$		-32.91	10.63		-349.8
	$-[16+3.75]\left[\frac{(0.0625)(1231.12-1220.12)}{16+29+16}\right](21.25)(6)$		-26.63	10.63		-283.1
	$-(1/2)[29-4-3.75]\left[\frac{(0.0625)(1231.12-1220.12)}{16+29+16}\right](21.25)(6)$		-14.33	14.17		-203.1
			-73.87			-836.0
	$e = \frac{2979.6}{266.26} = 11.19$ ft					
Total		-82.00	266.26			2979.6

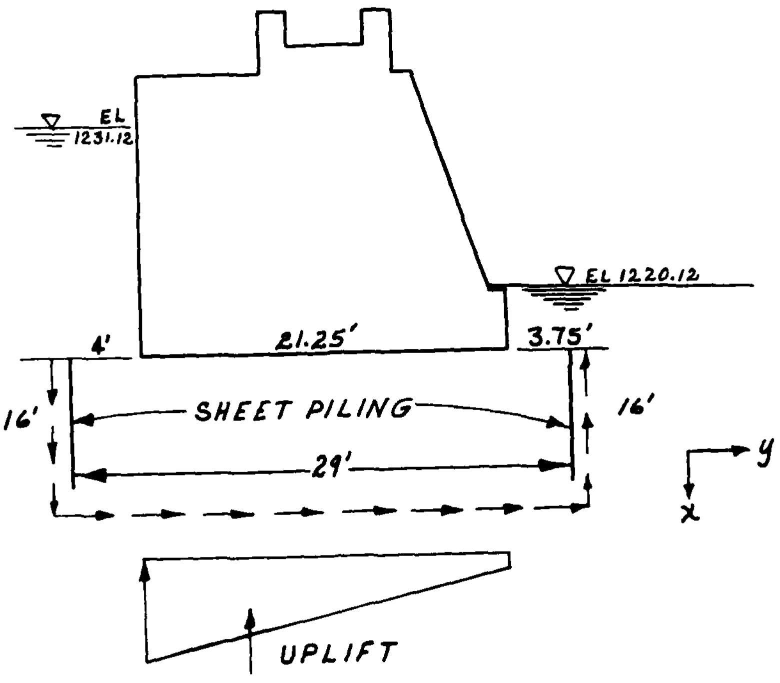


Figure 53. Pine River Dam, normal operation, reference el 1215.99, tailwater el 1220.12

Item	Factor	F_y (kips)	F_z (kips)	Arm_y (ft)	Arm_z (ft)	M_x (ft-k)
Loads	From normal operation calculations	-82.00	266.26			2979.6
P_{Truck}	H15-44 truck loading		24.00	13.00		312.0
Total		-82.00	290.26			3291.6

$$e = \frac{3291.6}{290.26} = 11.34 \text{ ft}$$

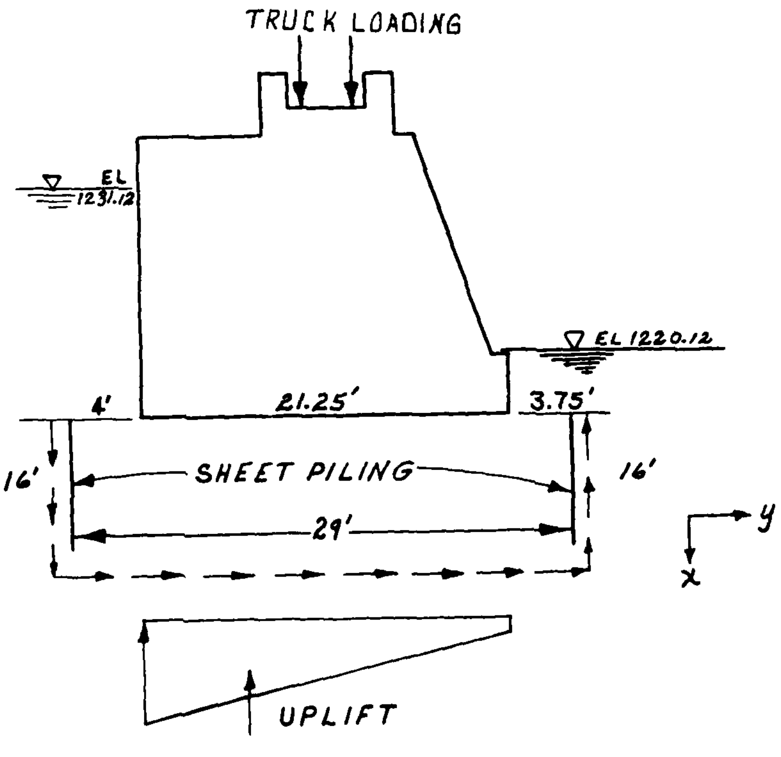


Figure 54. Pine River Dam, normal operation with truck loading (H15-44), reference el 1215.99, tailwater el 1220.12

Item	Factor	F_y (kips)	F_z (kips)	Arm_y (ft)	Arm_z (ft)	M_x (ft-k)
Loads	From normal operation calculations	-82.00	266.26			2979.6
Earthquake:						
Pe_1	(0.025)(340.13)	-8.50			10.49	-89.2
Pe_2	$(2/3)(51)(0.025)(15.13)^2(12)(1/1000)$	-2.33			6.05	-14.1
	$e = \frac{2876.3}{266.26} = 10.80$ ft					
Total		-92.83	266.26			2876.3

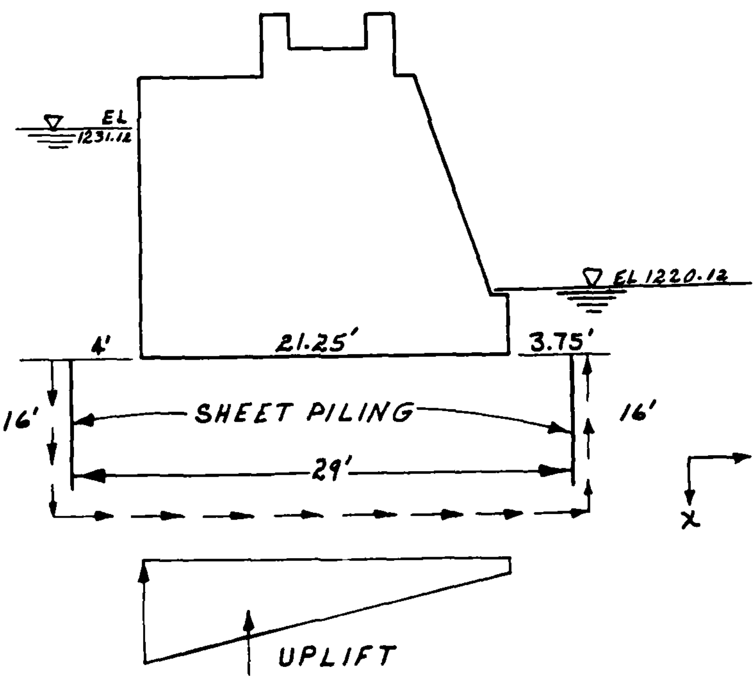


Figure 55. Pine River Dam, normal operation with earthquake, reference el 1215.99, tailwater el 1220.12

Item	Factor	F_y (kips)	F_z (kips)	Arm_y (ft)	Arm_z (ft)	M_x (ft-k)
Loads	From normal operation calculations	-82.00	266.26			2979.6
P_{Ice}	(1)(5)(12)	-60.00			14.63	-877.8
Total		-142.00	266.26			2101.8

$$e = \frac{2101.8}{266.26} = 7.89 \text{ ft}$$

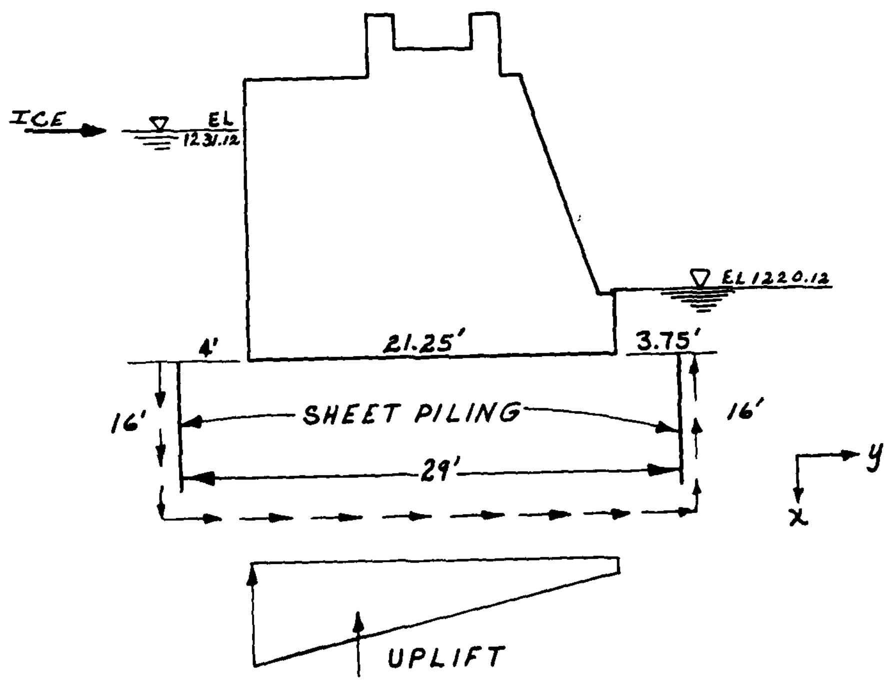


Figure 56. Pine River Dam, normal operation with ice, reference el 1215.99, tailwater el 1220.12

Item	Factor	F_y (kips)	F_z (kips)	Arm_y (ft)	Arm_z (ft)	M_x (ft-k)
W_{Conc}	(0.15)(1239.4-1235.82)(2)(12)		12.89	13.00		167.6
	(0.15)(1236.22-1235.22)(10)(12)		9.00	13.00		117.0
	(0.15)(1235.82-1234.32)(12.44)(6)		16.79	12.78		214.6
	(0.15)(1234.32-1231.32)(13.37)(6)		36.10	12.32		444.8
	$-(0.15)(1/2)(\pi)(3)^2(13.37)$		-28.35	12.32		-349.3
	(0.15)(1235.82-1214.32)(14.38)(6)		278.25	14.06		3912.2
	-(0.15)(1/2)(1235.82-1219.82)(6.62)(6)		47.66	4.66		222.1
	(0.15)(1219.82-1214.32)(6.87)(6)		34.01	3.44		117.0
	$-(0.15)(1/2)(\pi)(1)^2(8)$		-1.88	10.04		-18.9
	-(0.15)(1233.32-1219.82)(8)(2)		-32.40	7.56		-244.9
			372.06			4582.2
$P_{Headwater}$	$-(0.0625)(1/2)(1231.12-1214.32)^2(12)$	-105.84			5.60	-592.7
$P_{Tailwater}$	$(0.0625)(1/2)(1220.12-1214.32)^2(12)(0.6)$	7.57			1.93	14.6
Uplift	(0.0625)(1220.12-1214.32)(21.25)(6)		-46.22	10.63		-491.3
	$-[16+3.75][\frac{(0.0625)(1231.12-1220.12)}{16+29+16}](21.25)(6)$		-26.63	10.63		-283.1
	$-(1/2)[29-4-3.75][\frac{(0.0625)(1231.12-1220.12)}{16+29+16}](21.25)(6)$		-14.33	14.17		-203.1
			-87.18			-977.5
Total		-98.27	284.88			3026.6

$$e = \frac{3026.6}{284.88} = 10.62 \text{ ft}$$

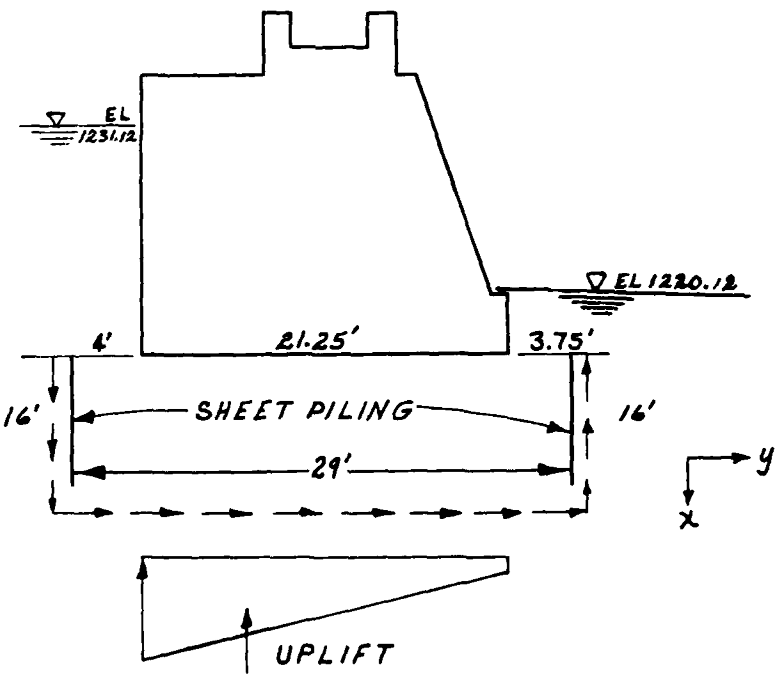


Figure 57. Pine River Dam, normal operation, reference el 1214.32, tailwater el 1220.12

Item	Factor	F_y (kips)	F_z (kips)	Arm_y (ft)	Arm_z (ft)	M_x (ft-k)
Loads	From normal operation calculations	-98.27	284.88			3026.6
P_{Truck}	H15-44 truck loading		24.00	13.00		312.0
Total		-98.27	308.88			3338.6

$$e = \frac{3338.6}{308.88} = 10.81 \text{ ft}$$

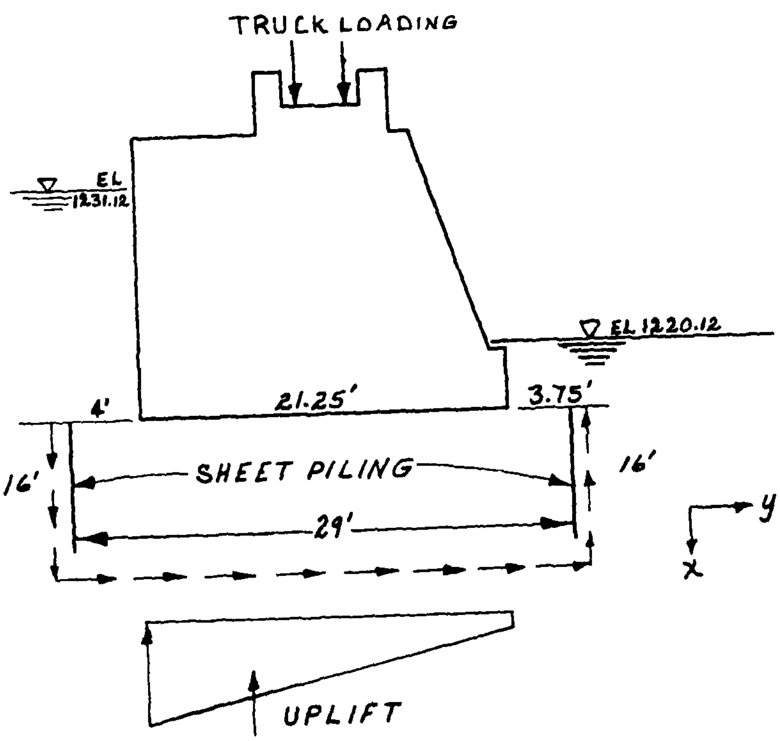


Figure 58. Pine River Dam, normal operation with truck loading (H15-44), reference el 1214.32, tailwater el 1220.12

Item	Factor	F_y (kips)	F_z (kips)	Arm_y (ft)	Arm_z (ft)	M_x (ft-k)
Loads	From normal operation calculations	-98.27	284.88			3026.6
Earthquake:						
Pe_1	(0.025)(372.06)	-9.30			11.19	-104.1
Pe_2	$(2/3)(51)(0.025)(16.80)^2(12)(1/1000$	-2.88			6.72	-19.4
Total		-110.45	284.88			2903.1

$$e = \frac{2903.1}{284.88} = 10.19 \text{ ft}$$

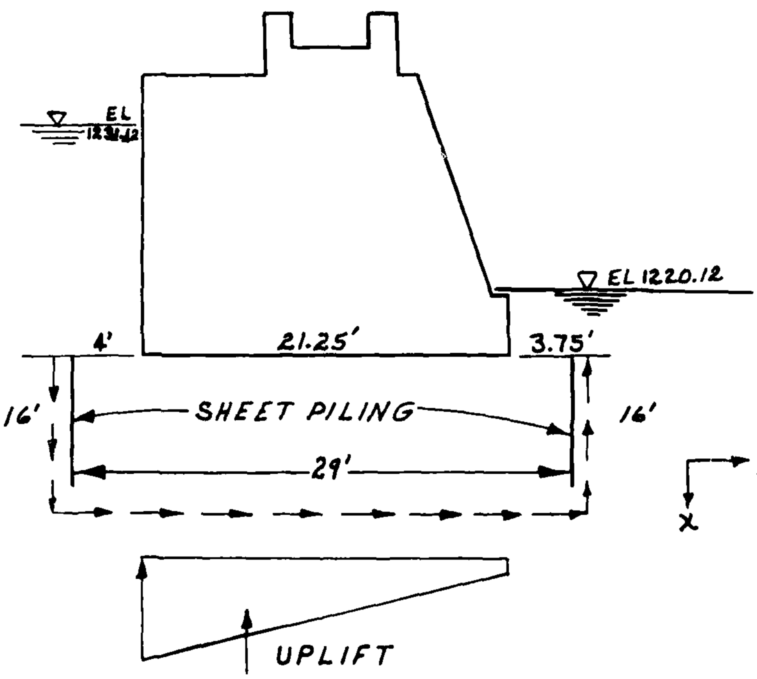


Figure 59. Pine River Dam, normal operation with earthquake, reference el 1214.32, tailwater el 1220.12

Item	Factor	F_y (kips)	F_z (kips)	Arm_y (ft)	Arm_z (ft)	M_x (ft-k)
Loads	From normal operation calculations	-98.27	284.88			3026.6
P_{Ice}	(1)(5)(12)	-60.00			16.30	-978.0
Total		-158.27	284.88			2048.6

$$e = \frac{2048.6}{284.88} = 7.19 \text{ ft}$$

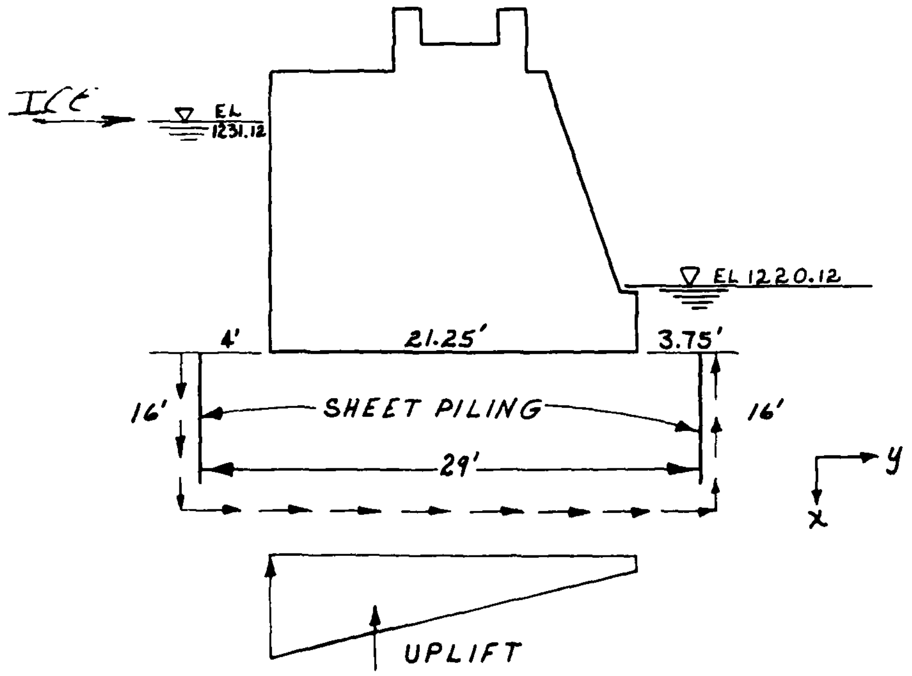


Figure 60. Pine River Dam, normal operation with ice, reference el 1214.32, tailwater el 1220.12

LOCATION OF CENTROID OF PILE GROUP

$$\bar{Y} = \frac{2[1.75 + 17.25] + 3.75 + 7.75 + 11.5 + 14.75}{8}$$

$$\bar{Y} = 9.47 \text{ ft}$$

$$\bar{X} = (1/2)(6) = 3 \text{ ft}$$

MOMENT OF INERTIA OF PILE GROUP ABOUT CENTROID OF PILE GROUP

$$I = I_o + Ad^2$$

$$I_{xx} = 2[(9.47 - 1.75)^2 + (9.47 - 17.25)^2] + (9.47 - 3.75)^2 + (9.47 - 7.75)^2 + (9.47 - 11.5)^2 + (9.47 - 14.75)^2$$

$$I_{xx} = 307.93 \text{ ft}^4$$

$$I_{yy} = [(4)(3)^2]$$

$$I_{yy} = 36.00 \text{ ft}^4$$

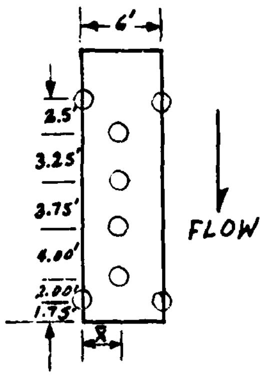


Figure 61. Moment of inertia of pile group (8 piles)

LOCATION OF CENTROID OF PILE GROUP

$$\bar{Y} = \frac{2[1.75 + 5.75 + 9.75 + 13.25 + 17.25] + 3.75 + 7.75 + 11.5 + 14.75}{14}$$

$$\bar{Y} = 9.52 \text{ ft}$$

$$\bar{X} = (1/2)6 = 3 \text{ ft}$$

MOMENT OF INERTIA OF PILE GROUP ABOUT CENTROID OF PILE GROUP

$$I = I_o + Ad^2$$

$$I_{xx} = 2[(9.52 - 1.75)^2 + (9.52 - 5.75)^2 + (9.52 - 9.75)^2 + (9.52 - 13.25)^2 + (9.52 - 17.25)^2] + (9.52 - 3.75)^2 + (9.52 - 7.75)^2 + (9.52 - 11.5)^2 + (9.52 - 14.75)^2$$

$$I_{xx} = 364.31 \text{ ft}^4$$

$$I_{yy} = [(10)(3)^2]$$

$$I_{yy} = 90 \text{ ft}^4$$

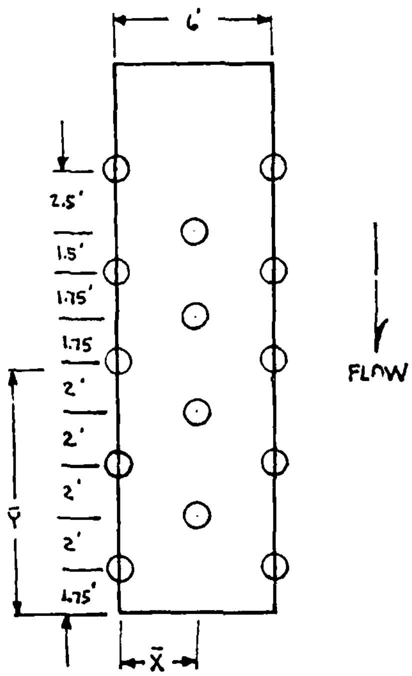


Figure 62. Moment of inertia of pile group (14 piles)

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STRUCTURAL STABILITY EVALUATION, PINE RIVER DAM.(U)
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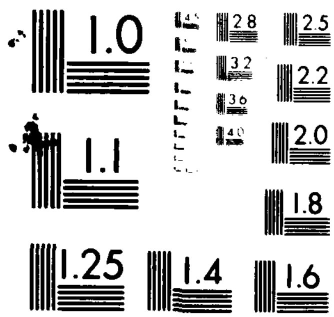

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

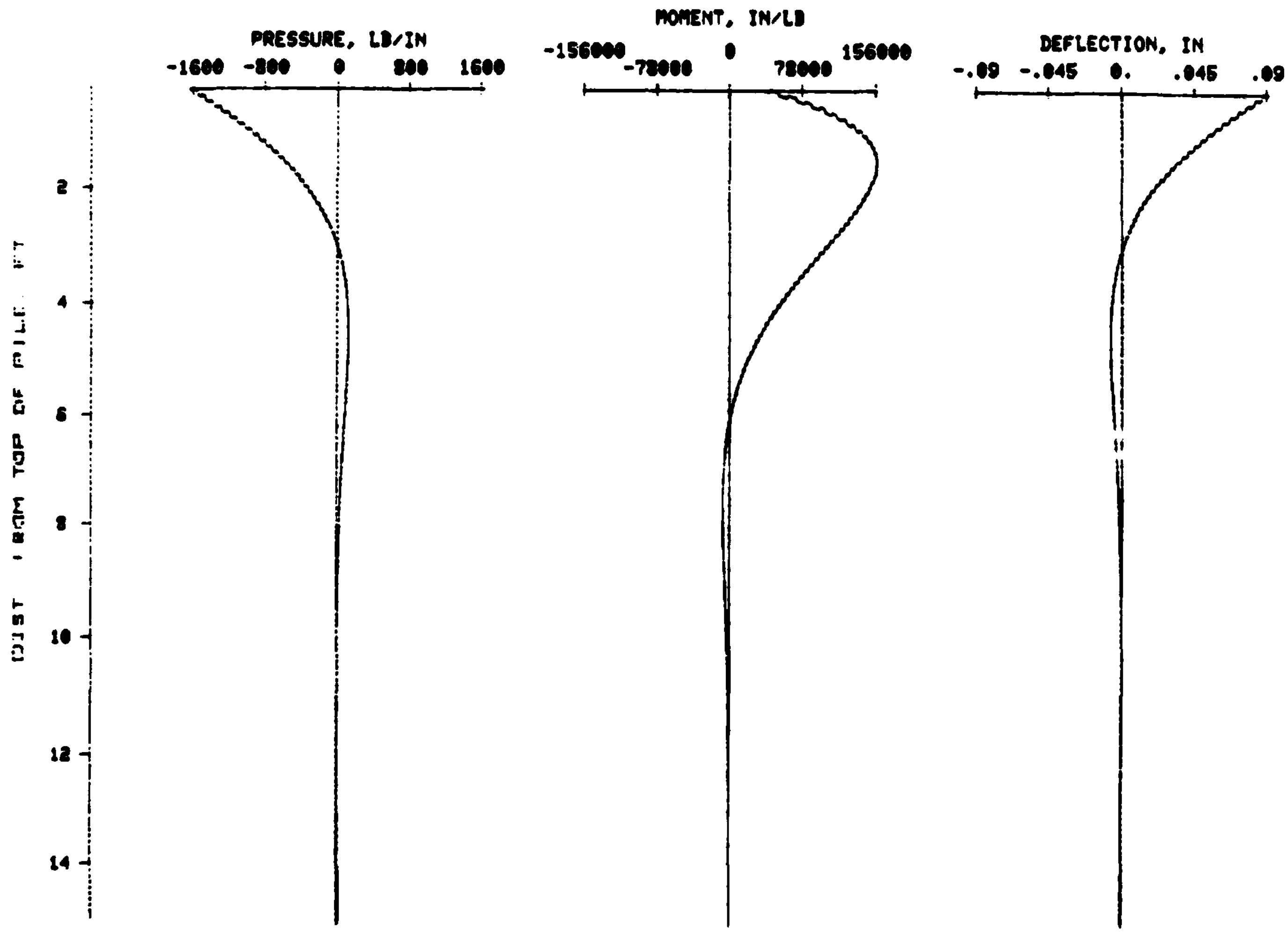

Figure 63. Pressure, moment, and deflection for the most critical loading of normal operation with ice

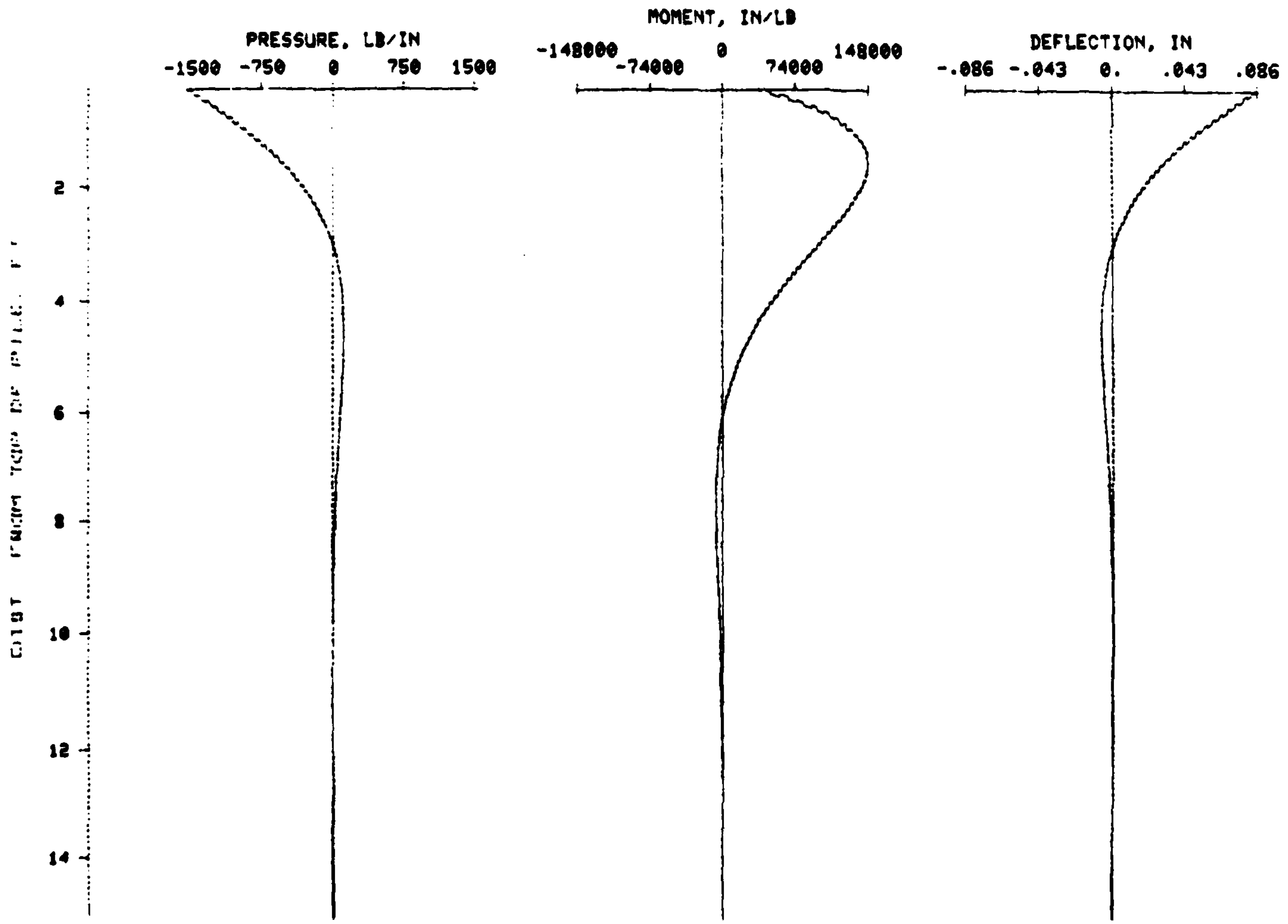

Figure 64. Pressure, moment, and deflection for the most critical loading of high-water condition

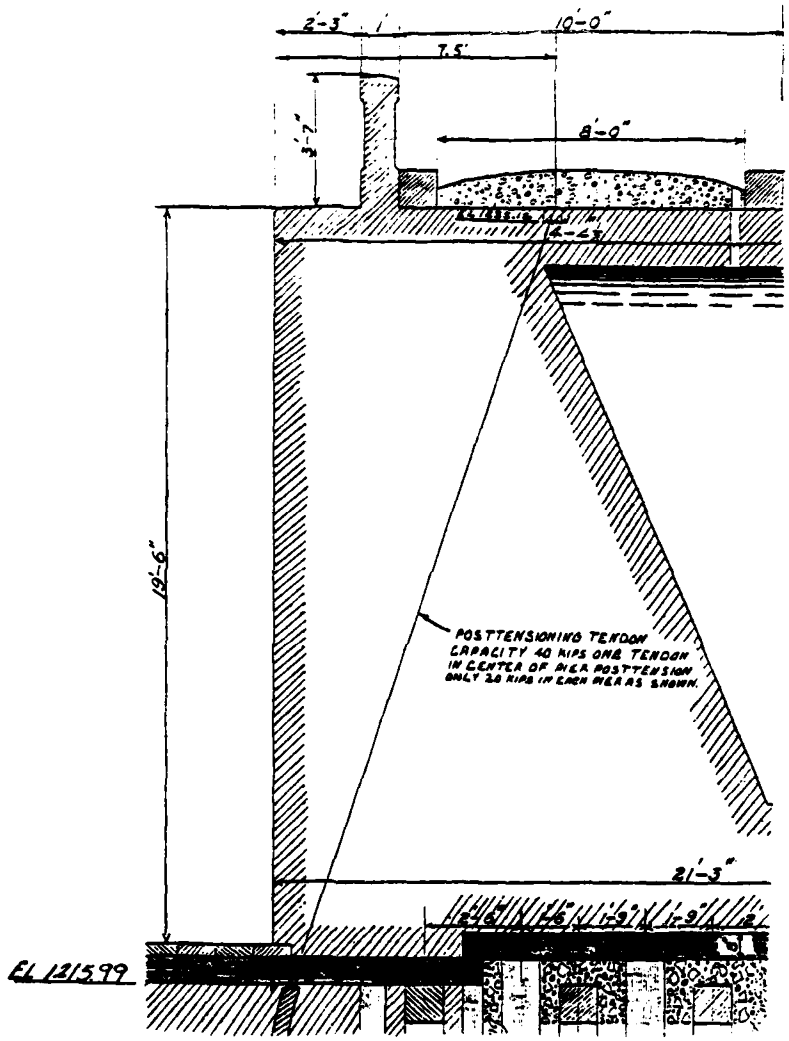

Figure 65. Placement of posttensioning

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Pace, Carl E.
Structural stability evaluation Pine River Dam / by Carl E. Pace (Structures Laboratory, U.S. Army Engineer Waterways Experiment Station). -- Vicksburg, Miss. : The Station ; Springfield, Va. : available from NTIS, 1981.
95 p. in various pagings : ill. ; 27 cm. -- (Miscellaneous paper / U.S. Army Engineer Waterways Experiment Station ; SL-81-31)
Cover title.
"October 1981."
"Prepared for U.S. Army Engineer District, St. Paul under Intra-Army Order No. NCS-1A-78-75."
Final report.
Bibliography: p. 30.

1. Concrete--Testing. 2. Concrete dams. 3. Pine Tree Dam (Minn.) 4. Structural stability. I. United States. Army. Corps of Engineers. St. Paul District. II. U.S. Army Engineer Waterways Experiment Station. Structures Laboratory. III. Title. IV. Series: Miscellaneous paper (U.S. Army Engineer Waterways

Pace, Carl E.
Structural stability evaluation Pine River Dam : ... 1981.
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Experiment Station) ; SL-81-31.
TA7.W34m no.SL-81-31